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Ontario
Ministry of the Environment

# Storm Water Management for Nepean, Merivale Area

## Research Report No. 89

STANDARDS DEVELOPMENT BRANCH OMOE
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## Research Program for the Abatement of Municipal Pollution under Provisions of the Canada-Ontario Agreement on Great Lakes Water Quality

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Storm water management for
Nepean, Merivale area.

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# STORM WATER MANAGEMENT FOR

# NEPEAN, MERIVALE AREA

Gore and Storrie Ltd.
Consulting Engineers

RESEARCH PROGRAM FOR THE ABATEMENT
OF MUNICIPAL POLLUTION UNDER THE
PROVISIONS OF THE CANADA-ONTARIO
AGREEMENT ON GREAT LAKES WATER QUALITY

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## ABSTRACT

A detailed analysis of the quantity and quality of storm runoff from the Merivale area of Nepean, Ontario, to the Rideau River was carried out. The Merivale area watershed comprises 1300 acres of industrial (705 acres), residential (135 acres) and open land (460 acres). An open ditch storm drainage system collects and transports runoff from the originally developed areas to Nepean Creek, and ultimately to the Rideau River. The newly developing area of Merivale Acres Industrial Colony is serviced by storm sewers which drain to a 120-inch diameter main trunk sewer routed through the development area. A main trunk and local sewer system plan has been developed to service the entire area. The Rideau River has been identified by the Ontario Ministry of Environment as a particularly sensitive watercourse with a limited ability to assimilate polluting discharges.

The Storage, Treatment Overflow Runoff Model (STORM), developed by the U.S. Army Corps of Engineers, was used in this analysis for continuous simulation of rainfall and runoff based on hourly precipitation data. The Storm Water Management Model (SWMM), as developed by the U.S. Environmental Protection Agency, was used to simulate specific storm events. These models generated quantity and quality information for storm runoff from the area. Based on this information, three storm water management control and treatment alternatives were prepared. Estimated costs for the proposed control and treatment alternatives are provided.

## RESUME

Cet ouvrage présente une analyse quantitative et qualitative détaillée des eaux pluviales, depuis le secteur Merivale, à Nepean (Ontario), jusqu'à la rivière Rideau. Le bassin de ce secteur comprend une zone de 1300 acres, dont 705 de terrains industriels, 135 de terraines résidentiels et 460 de terrains vagues. Un réseau de fossés achemine les eaux de pluie provenant des zones aménagées vers le ruisseau Nepean, lequel se jette dans la rivière Rideau. Dans le nouveau secteur mis en valeur par la Merivale Acres Industrial Colony, un réseau évacue les eaux pluviales vers une conduite maîtresse de 120 pouces de diamètre. Celle-ci traverse ledit secteur et se raccorde au collecteur principal. Un réseau de canalisations raccordées à un collecteur local permettra de répondre aux besoins du secteur tout entier. Or, le ministère ontarien de l'Environnement considère la rivière Rideau particulièrement vulnérable, à cause de son faible seuil de tolérance de la pollution.

Les auteurs de l'analyse ont utilisé, pour la simulation d'averses continues et l'écoulement des eaux de ruissellement, basée sur les précipitations horaires, le Storage, Treatment, Overflow, Runoff Model (STORM) mis au point par les ingénieurs des Forces armées américaines. Quant à la simulation de cas précis, ils ont utilisé le Storm Water Management Model (SWMM) conçu par l'EPA, aux États-Unis. Grâce à ces modèles, on a pu recueillir de nombreuses données valables sur l'écoulement des eaux pluviales dans ce secteur et, ce faisant, élaborer trois options de gestion et d'épuration des eaux pluviales. L'ouvrage présente les coûts estimés de ces trois projets.

TABLE OF CONTENTS


| | | Page |
|---|---|---|
| ABSTRACT | | i |
| TABLE OF CONTENTS | | iii |
| | List of Figures | v |
| | List of Tables | vi |
| 1 | INTRODUCTION | 1 |
| 1.1 | Study Area | 1 |
| 1.2 | Purpose of Study | 3 |
| 1.3 | Study Funding and Organization | 3 |
| 1.4 | Terms of Reference | 4 |
| 2 | STORM SEWER SYSTEM | 5 |
| 2.1 | Existing Storm Sewer System | 5 |
| 2.2 | Minor and Major Drainage Systems | 5 |
| 2.3 | Proposed Storm Sewer System | 5 |
| 2.4 | Present Storm Sewer Design Practice | 6 |
| 3. | RIDEAU RIVER | 10 |
| 3.1 | Watershed and Flows | 10 |
| 3.2 | Water Quality | 11 |
| 3.2.1 | General | 11 |
| 3.2.2 | Rideau River | 11 |
| 3.3 | Ministry of the Environment Guidelines | 12 |
| 4 | STORM WATER MANAGEMENT TECHNIQUES | 15 |
| 4.1 | General | 15 |
| 4.2 | Storm Water Runoff Controls | 15 |
| 4.2.1 | Surface storage | 16 |
| 4.2.2 | Subsurface storage | 22 |
| 4.3 | Nepean Development Controls | 25 |
| 5. | ANALYSIS PROCEDURES AND RESULTS | 29 |
| 5.1 | Model Simulation | 29 |
| 5.1.1 | STORM | 29 |

TABLE OF CONTENTS (CONT'D)


| | | Page |
|---|---|---|
| 5.1.2 | SWMM | 30 |
| 5.2 | Rainfall and Design Storm Analysis | 35 |
| 5.3 | Storm Water Management Surface Storage Controls | 39 |
| 5.3.1 | Surface storage options | 39 |
| 5.3.2 | Surface storage analysis results | 43 |
| 5.4 | Subsurface Storage/Treatment Controls | 45 |
| 5.4.1 | Storage size requirements | 45 |
| 5.4.2 | Storage/treatment analysis results | 51 |
| 5.5 | Quality Parameters Modelled and Accuracy of Predictions | 63 |
| 5.5.1 | General | 63 |
| 5.5.2 | Suspended solids | 65 |
| 5.6 | Accuracy of Model Predictions | 66 |
| 6. | ESTIMATED COSTS | 67 |
| 6.1 | Surface Storage and Management | 67 |
| 6.2 | Subsurface Storage/Treatment | 67 |
| 7 | CONCLUSIONS AND RECOMMENDATIONS | 69 |

## LIST OF FIGURES


| Figure | | Page |
|---|---|---|
| 1 | Storm Drainage Master Plan | 2 |
| 2 | Rainfall Intensity-Duration Curves | 7 |
| 3 | Rainfall Detention Ponding Ring for Flat Roofs | 18 |
| 4 | Controlled Rate Inlets | 19 |
| 5 | 120-inch Diameter Trunk Storm Sewer | 23 |
| 6 | Control Chamber | 24 |
| 7 | Storm Water Management Model Structure | 32 |
| 8 | Rainfall Hyetographs | 36 |
| 9 | Outflow Hydrographs Without Retention | 38 |
| 10 | Typical Storm Drainage Area (72 acres) Hydrographs | 41 |
| 11 | Annual Number of Bypass Conditions vs. Proposed Retention Pond Size | 44 |
| 12 | One-year Synthetic Storm Hydrographs | 46 |
| 13 | BOD Pollutographs | 47 |
| 14 | Suspended Solids Pollutographs | 48 |
| 15 | Annual Number of Bypass Conditions vs. Proposed Storage Size | 50 |
| 16 | Hydrographs Through Treatment | 58 |
| 17 | BOD Pollutographs Through Treatment | 59 |
| 18 | Suspended Solids Pollutographs Through Treatment | 60 |

## LIST OF TABLES


| Table | | Page |
|---|---|---|
| 1 | Township of Nepean - Values for Runoff Coefficient "C" | 9 |
| 2 | Total Loadings of Pollutants in Washoff | 37 |
| 3 | Synthetic One-Year Storm Summary | 45 |
| 4 | Summary of Average Annual Washoff from Watershed | 52 |
| 5 | Summary of Average Annual Flow and Quality of Runoff Considering Managed Runoff and Treatment | 54 |
| 6 | Array of Significant Storms | 56 |
| 7 | Summary of Flow and Quality Results Through Treatment (Contents of Storage Tank Retained) | 61 |
| 8 | Summary of FLow and Quality Results Through Treatment (Contents of Storage Tank Discharged) | 62 |
| 9 | Summary of Flow and Quality Results Through Treatment (Sedimentation with Chemical Coagulation) | 63 |
| 10 | Summary of Capital and Operating Costs for Treatment Facilities | 68 |

# 1 INTRODUCTION

## 1.1 Study Area

The Merivale watershed in the Nepean*, Ontario, comprises a total area of approximately 1300 acres.

Land use within the watershed varies and includes industrial, residential and open space. Most of the land within the watershed is industrial and comprises approximately 705 acres. There are two residential subdivisions: Merivale Gardens which comprises approximately 75 acres, and the Tanglewood subdivision of approximately 60 acres. The remaining 460 acres are designated as open space. Open space designations include parkland within the residential areas, hydro corridors and the 390-acre National Capital Commission Greenbelt lands around the Merivale Gardens subdivision. The boundaries of the watershed and the land use and zoning are shown on Figure 1.

The residential land within the watershed is fully developed and much of the industrial land is also developed. Some industrial land remains undeveloped or is in various stages of development. Most notable is the Merivale Acres Industrial Colony located on the north side of the C.N.R. right-of-way, between Merivale Road and Highway No. 16. This area (200 acres) is being developed by the Township of Nepean as an industrial park. The second major area of undeveloped industrial land, which comprises approximately 100 acres, is located west of Merivale Road and is generally bounded by Merivale Road on the East, the NCC Greenbelt on the west, Enterprise Avenue on the south and the Tanglewood residential subdivision on the north. This land is privately-owned and development proposals are being formulated.

The Merivale area is very flat and thus has generally very poor drainage. Storm sewers have not yet been provided to the originally developed areas and storm drainage is presently directed through an open ditch system which transports flows from the area to the Nepean Creek and ultimately to the Rideau River. Because of the very small grades

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* Nepean became a City November 24, 1978. This study was completed previous to that date and Nepean is referred to as a Township throughout most of this report.

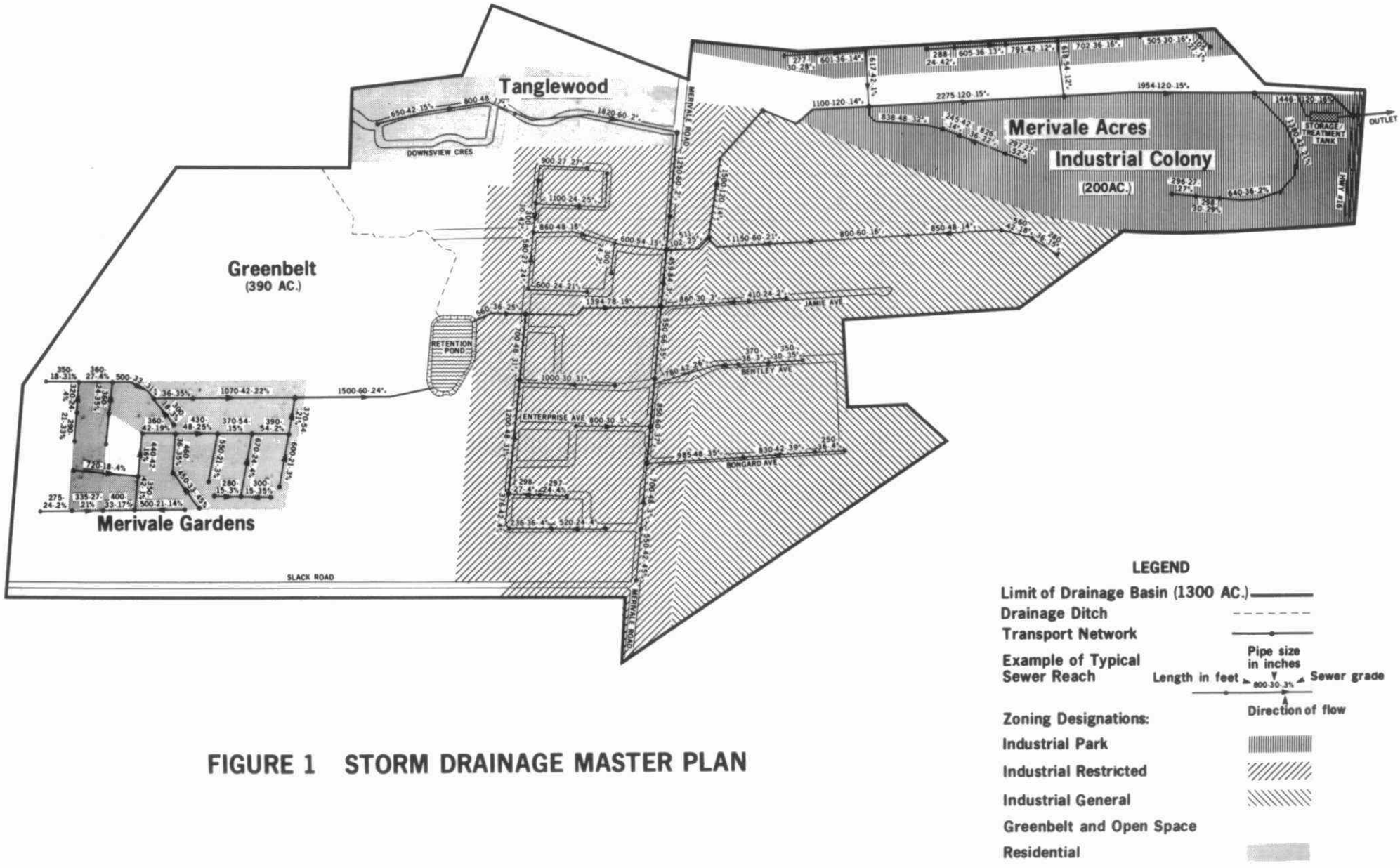


**FIGURE 1 STORM DRAINAGE MASTER PLAN**

available, the open ditch system is less than satisfactory. Unwanted ditch ponding creates nuisance weed growth and maintenance requirements are extremely high.

## 1.2 Purpose of Study

In view of the drainage problems and the increasing pressures for industrial development within the area, the Township of Nepean initiated a plan to provide both storm and sanitary sewer services for the Merivale area.

The trunk storm sewer system which was developed to service the area is outlined on Figure 1. The trunk sewer follows an alignment parallel to and south of the Nepean Creek, traversing in a west to east direction through the Merivale Acres Industrial Colony and discharging into the Rideau River.

The detrimental effects of storm runoff on receiving waters have been recognized for some years. Only recently, however, has detailed attention been given to this problem and possible remedies.

The Rideau River, which receives the runoff from the Merivale watershed, has a limited capacity to assimilate pollutant loadings. Recognizing this, the Ontario Ministry of the Environment (MOE), in December of 1973, advised the Regional Municipality of Ottawa-Carleton of its concern about the discharge of runoff from new developments within the Rideau River watershed. The Ministry stated, at that time, that further development along the Rideau River "will have to be based on the principle of no deterioration to the existing quality of the water-courses". In addition, the Ministry stated that large new developments "will require a report on the effect of storm drainage on the area water-courses and the method of proposed control".*

The development plans for the Merivale Area, and the policy stipulated by the Ministry, as outlined above, required that a detailed analysis be undertaken to determine the quantity and quality of runoff from the Merivale area to the Rideau River, and to investigate methods for management of the runoff to meet the quality objectives of the Ministry of the Environment.

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* Letter dated December 20, 1973 from Ontario Ministry of the Environment to Regional Municipality of Ottawa-Carleton.

## 1.3 Study Funding and Organization

This study was initiated by the Township of Nepean in conjunction with the Regional Municipality of Ottawa-Carleton. Fifty percent of the funding for the study was provided jointly by Environment Canada and the Ontario Ministry of the Environment (MOE) under the Canada-Ontario Agreement on Great Lakes Water Quality, through the Urban Drainage Subcommittee. The remaining 50 percent was provided by the Regional Municipality of Ottawa-Carleton.

The study was undertaken under the direction of a steering committee composed of representatives of:

- The Regional Municipality of Ottawa-Carleton,
- The Corporation of the Township of Nepean,
- Environment Canada,
- The Ministry of the Environment, Ontario.

## 1.4 Terms of Reference

The terms of reference for the study were described in the engineering agreement for the work between the Regional Municipality of Ottawa-Carleton and Gore and Storrie Limited and are summarized as follows:

1) The Consultants shall analyse the watershed utilizing the full capabilities of both the computer model SWMM (excluding the receiving block) as developed for the United States Environmental Protection Agency, with such modifications as are developed for and available from the Canadian Government as well as the computer model STORM as developed by the U.S. Army Corps of Engineers.
2) The Consultants shall develop an estimate of both quantity and quality of runoff for existing and future development.
3) The Consultants shall develop an overall storm drainage system, integrating conveyance and treatment components to meet with Ministry of the Environment, Environment Canada and Corporation approval. The project will examine all viable options, possible phasing, capital and operating expenses and will make recommendations based on the analysis of the above options.

## 2 STORM SEWER SYSTEM

### 2.1 Existing Storm Sewer System

The Merivale watershed, which was described previously, comprises approximately 1300 acres and is outlined in Figure 1. An open ditch system collects and transports runoff from the originally developed areas to the Nepean Creek and ultimately to the Rideau River. The total watershed area of the Nepean Creek is approximately 2700 acres, including the 1300-acre Merivale watershed.

The newly developed Merivale Acres Industrial Colony is serviced with storm sewers which drain to a 120-inch diameter main trunk storm sewer which is routed through the area. The trunk sewer alignment is shown on Figure 1 and is designed to receive flows from and to facilitate ultimate development of the entire watershed.

### 2.2 Minor and Major Drainage Systems

The storm sewer systems are designed to contain a storm with a relatively short return frequency. Most municipalities, including the Township of Nepean, use a five-year return design storm. This pipe system is commonly referred to as a "minor" drainage system.

In planning and design of drainage systems to service any development, proper consideration must be given also to the excess runoff which may result from a very severe storm which cannot be handled by the minor drainage system. When the capacity of the pipe system is exceeded, the excess runoff will attempt to flow overland following routes of natural watercourses. Therefore, it is important that final grading within the development make provision for this occurrence. This latter overland flow system is generally referred to as a "major" drainage system.

This study assessed the details of the minor system only, and its capabilities for handling a design storm as well as more frequent and average rainfalls. Major storms were not assessed for quality aspects.

### 2.3 Proposed Storm Sewer System

Figure 1 shows the storm drainage master plan for the Merivale area. The main trunk sewer is proposed to extend southerly from the

Merivale Acres Industrial Colony to service the Merivale sector of the industrial area along Merivale Road. It is proposed that the trunk sewer will ultimately extend westerly to provide storm sewer services to the Merivale Gardens subdivision, when required.

## 2.4 Present Storm Sewer Design Practice

The existing local and trunk storm sewers servicing the Merivale Acres Industrial Colony, as well as all proposed sewers shown on Figure 1, have been sized for capacity requirements in accordance with the Township of Nepean's present standard criteria for storm sewer design. These criteria utilize the "Rational Method" for calculating runoff and the Manning Formula for hydraulic design of sewers.

The Rational Method has been in widespread use in North America for the design of urban storm sewer systems for many years. It is an empirical method that uses rainfall intensity, contributing land area and ground surface characteristics to calculate storm wastewater peak flow rates. The peak flow, as calculated, is used in the sizing of the storm sewers.

The Rational Method is of the following form:

$$Q\ (cfs) = AiC$$

where: A = contributary area (acres),
i = rainfall intensity (inches per hour),
C = runoff coefficient (dimensionless, generally varying from 0.1 to 1.0),
Q = peak flow in cfs.

Of the above factors, only the area (A) can be determined precisely. It is generally measured from plans and represents the gross area that may be expected to contribute runoff to the sewer under design.

Rainfall intensity (i) is determined from rainfall intensity-duration curves that are a statistical interpretation of rain occurrences made from recording rain gauge station data. They represent the maximum intensity that is expected to occur for a particular length of time (duration) within a given return period. Curves developed for 2, 5, 10, 50 and 100-year return periods are shown on Figure 2.

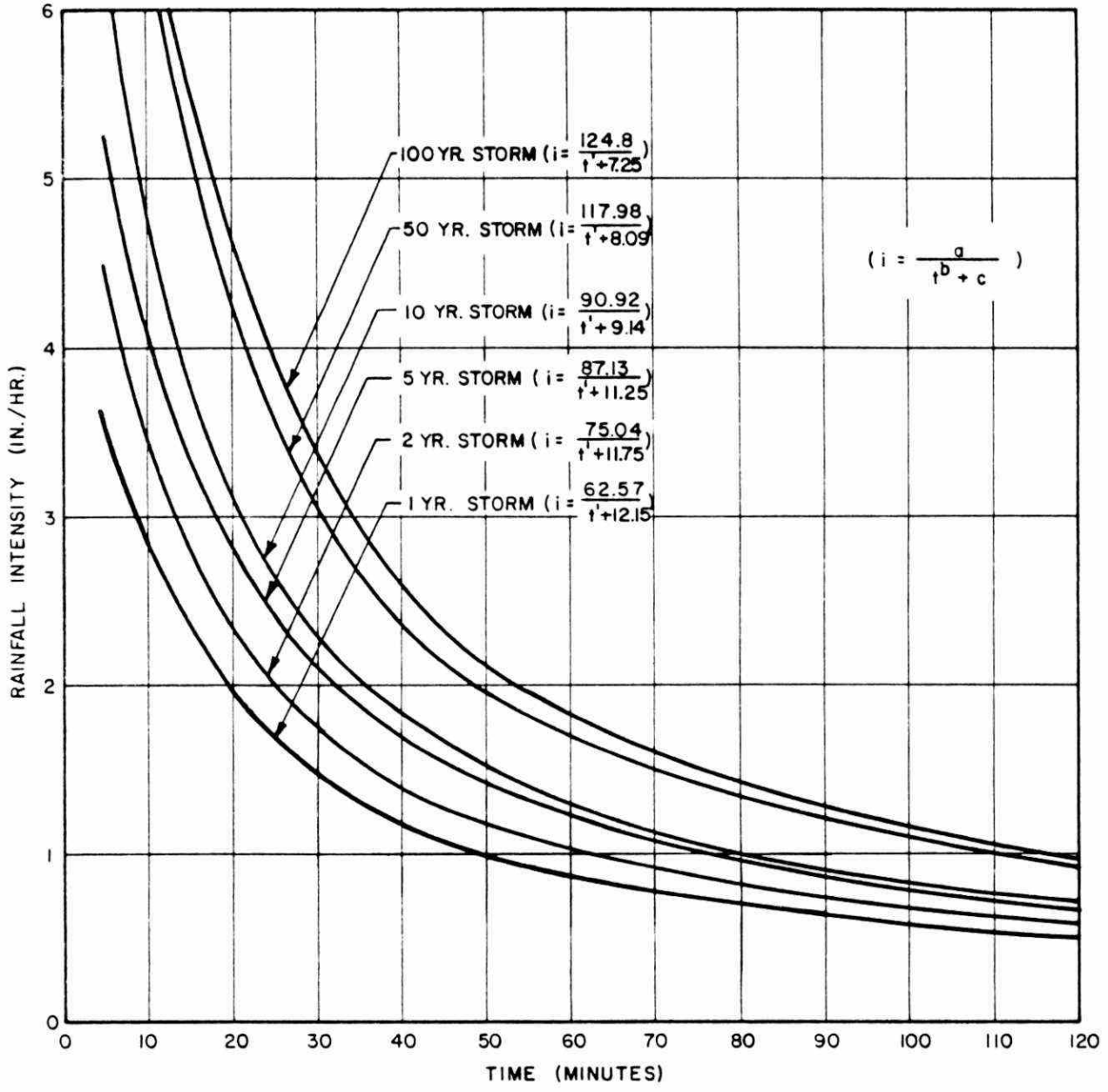


FIGURE 2. RAINFALL INTENSITY-DURATION CURVES

The two-year to 100-year curves shown on Figure 2 were produced by the Atmospheric Environment Service of Environment Canada on the basis of a statistical analysis of rainfall records for the Ottawa Central Experimental Farm for a period of 17 years of rainfall records.

The rainfall intensity used for design is established by first selecting the return period, then determining the time of concentration (tc), which is the sum of the inlet or overland flow time plus the time for flow in the sewer to the point under consideration.

The Township of Nepean present criteria use the five-year rainfall intensity-duration curve for design, and a maximum intensity of 3.4 inches per hour, corresponding to a time of concentration of 15 minutes. An inlet time of 15 minutes was used for design of the Merivale area storm sewers.

The runoff coefficient (C) is the proportion of the rainfall that is expected to enter the sewer system and is an empirical function related to the characteristics of the ground surface. A number of factors, including infiltration, surface ponding, evaporation, amount of impervious surface and surface slope, all affect the runoff coefficients.

Table 1 summarizes the current Township of Nepean practice for selection of runoff coefficients. The application of these coefficients is usually a matter of personal judgement by the designer.

The Rational formula gives an estimate of the peak flow only, which is used to size the pipe network. A hydrograph of flow is not calculated and, therefore, the effects of storage of storm water either above or below ground cannot be adequately assessed.

TABLE 1. TOWNSHIP OF NEPEAN - VALUES FOR RUNOFF COEFFICIENT "C"

| Topography and Vegetation | | Soil Texture | | |
|---|---|---|---|---|
| | | Open Sandy Loam | Clay and Silt Loam | Tight Clay |
| | Woodland | | | |
| Flat | 0-5% Slope | 0.10 | 0.30 | 0.40 |
| Rolling | 5-10% Slope | 0.25 | 0.35 | 0.50 |
| Hilly | 10-30% Slope | 0.30 | 0.50 | 0.60 |
| | Pasture | | | |
| Flat | 0-5% Slope | 0.10 | 0.30 | 0.40 |
| Rolling | 5-10% Slope | 0.16 | 0.36 | 0.55 |
| Hilly | 10-30% Slope | 0.22 | 0.42 | 0.60 |
| | Cultivated | | | |
| Flat | 0-5% Slope | 0.30 | 0.50 | 0.60 |
| Rolling | 5-10% Slope | 0.40 | 0.60 | 0.70 |
| Hilly | 10-30% Slope | 0.52 | 0.72 | 0.82 |
| | Urban Areas | 30% Impervious | 50% Impervious | 70% Impervious |
| Flat | 0-5% Slope | 0.40 | 0.55 | 0.75 |
| Rolling | 5-10% Slope | 0.50 | 0.65 | 0.80 |

| Paved Areas | |
|---|---|
| Bituminous and Conc. Pavement | C = 0.90 - 1.00 |
| Densely Built - Paved Areas | C = 0.90 |
| Commercial and Factory Areas, Road Shoulders | C = 0.70 |
| Light Factories and Apartment Areas | C = 0.60 |
| Compact Residential Areas | C = 0.50 |
| Suburban Residential Areas | C = 0.30 - 0.40 |
| Gravel Surfaces | C = 0.25 - 0.70 |
| Parks, Railway Yards | C = 0.20 |

NOTE: Values of "C" can be combined to get an average for a watershed consisting of several distinct zones.

e.g., watershed consists of:

20% hilly woodland on sandy loam (C = 0.30),
30% rolling woodland on clay and silt loam (C = 0.35),
50% rolling Cultivated tight clay (C = 0.70).

Average "C" (0.2x0.30) + (0.3x0.35) + (0.5x0.70) = 0.52.

# 3 RIDEAU RIVER

## 3.1 Watershed and Flows

The watershed of the Rideau River, at the point of discharge of the Merivale trunk sewer, is approximately 1480 square miles, with headwaters in the Carnahan Lake area at the western extremity.

A number of urban communities are located within the watershed boundaries, including the Town of Smiths Falls, the Town of Perth, the Town of Kemptville, the Village of Manotick and others. Most of the area, however, is either undeveloped woodlands or agricultural land.

Although the watershed of the Rideau River is relatively large, its particular characteristics result in low base flows in the river throughout most of the year. Base flows in the Rideau River throughout the summer months generally average in the order of 300 cubic feet per second, measured at Ottawa. The peak flow from the Merivale trunk sewer for the design storm with a recurrence interval of once in five years is estimated at approximately 700 cfs. Peak flows in the Rideau River can, of course, be very much greater than the average summer base flows; the maximum peak flow recorded was approximately 21,400 cfs during the 1976 spring runoff. The previous peak flow recorded was approximately 19,000 cfs.

Because of the nature of the Rideau River watershed and its relative size compared to the Merivale watershed, peak flows generated from a particular storm would not be expected to occur simultaneously. Thus, the volume of flow in the Rideau River required for proper and adequate dilution of runoff from tributary drainage sources would not necessarily be available.

Long lasting high volume storms which are relatively uniformly distributed over the watershed may produce increased flows in the river, adequate for proper assimilation of runoff volumes from the development. Also, particularly severe but localized storms could produce large runoffs and create high peak flows in minor tributaries, such as the Merivale trunk sewer, without significantly affecting upstream flows in the Rideau River.

## 3.2 Water Quality

### 3.2.1 General

The Ontario Ministry of Health's water quality guidelines for bathing beaches, take into account:

a) sanitary survey,

b) medical considerations,

c) physical quality of water, and

d) bacterial guidelines.

The bacterial guidelines are briefly summarized as follows: The quality of bathing beach waters is considered impaired when the total coliform geometric mean exceeds 1000 per 100 ml and/or the fecal coliform geometric mean exceeds 100 per 100 ml. It is important that interpretations of total coliforms not be considered alone. A series of at least 10 samples per month per sampling location is recommended. More samples will be required when the beach is more frequently used or when the beach is subject to contamination.*

The Ontario Ministry of the Environment microbiological criteria for surface waters used for swimming are as follows:

> "Water used for body contact recreational activities should be free from pathogens including any bacteria, fungi or viruses that may produce enteric disorders or eye, ear, nose, throat and skin infections. Where ingestion is probable, recreational waters can be considered impaired when coliform, fecal coliform and/or enterococcus (fecal streptococcus) geometric mean density exceeds 1000, 100 and/or 20 per 100 ml in a series of at least 10 samples per month, including samples collected during weekend periods".**

### 3.2.2 Rideau River

The water quality of the Rideau River is considered to be generally good, particularly upstream of the City of Ottawa.

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* Water Quality Guidelines for Bathing Beaches, Ontario Ministry of Health, Toronto.

** Guidelines and Criteria for Water Quality Management in Ontario, Ontario Ministry of the Environment, Toronto, July 1974.

There is a significant algae problem in the river which is attributed to the nutrient contribution from upstream urban and agricultural developments, as well as from woodlands. Continuing efforts are being made by the Ontario Ministry of the Environment to control such problems. Upgraded sewage treatment plants with increased phosphorus removal facilities are being planned and installed in most urban centres.

Biochemical oxygen demand (BOD) values in the Rideau River are generally less than 4.0 mg/L. Mean dissolved oxygen concentrations are generally in excess of 5.0 mg/L, the MOE criteria for warm water fish.

The bacteriological quality of the Rideau River upstream of Hog's Back at Ottawa has generally been found to conform to Ontario Ministry of Health and MOE criteria. The swimming beach at Mooney's Bay has been open most of the time, with the approval of the Ontario Ministry of Health. There are occasions, however, particularly after heavy rainfalls, when the indicator values may exceed the specified limits.

It is the opinion of the Ontario Ministry of the Environment, however, that "the existing upstream quality can be maintained and that there will also be some further improvements providing there is adherence to an adequate development policy"*.

Swimming beaches along the downstream reaches of the river through the City of Ottawa have been closed for some years because of the adverse bacteriological quality of the water. Continuing efforts by the City of Ottawa and the Regional Municipality of Ottawa-Carleton to eliminate illegal cross-connections and to reduce combined sewer overflows have resulted in a much improved bacteriological water quality. The Regional Municipality is optimistic that the improving water quality in the river through the City of Ottawa will permit the re-opening of additional bathing beaches in the near future.

## 3.3 Ministry of the Environment Guidelines

Although the water quality of the Rideau River upstream of Hog's Back is relatively good, the low base flows in the river severely limit its ability to assimilate waste. In view of this and the statement

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* Letter dated December 20, 1973, from Ontario Ministry of the Environment to the Regional Municipality of Ottawa-Carleton.

by the Ministry that "further development will have to be based on the principle of no deterioration to the existing quality of local watercourses", the Ministry has formulated a set of guidelines as quality standards for storm water discharges to the Rideau River. These Guidelines were supplied by the Ministry to the steering committee guiding this study on January 7, 1976 and confirmed in writing to the Regional Municipality of Ottawa-Carleton by letter dated February 27, 1976.

The guidelines were set by the Ministry as a numerical interpretation of the principle of "no deterioration" stated above. The Ministry recognizes that these guidelines are subject to change if new information becomes available, and that the best means of obtaining additional information to update the guidelines would be to carry out a comprehensive water quality study of the Rideau River. A study of this nature would take into account water quality criteria, the effects of existing and proposed urbanization on water quality, upstream water quality effects and the cost and effects of various preventative and remedial measures to control pollution.

The guidelines suggested by MOE relate to both effluent quality objectives and receiving water quality objectives after mixing with the effluent. The quality objectives supplied by MOE were as follows:

Effluent Objectives

| | | |
|---|---|---|
| BOD | - | 5 to 6 mg/L |
| Total phosphorus | - | 1 mg/L |
| Chlorine | - | 0.1 mg/L |
| Total coliform | - | 1000/100 ml |
| Fecal coliform | - | 100/100 ml |
| Fecal streptococci | - | 20/100 ml |
| SS | - | maximum removal |

Receiving Water Criteria

| | | |
|---|---|---|
| BOD | - | 4 mg/L |
| Chlorine | - | 0.02 mg/L |
| Total coliform | - | 1000/100 ml |
| Fecal coliform | - | 100/100 ml |
| Fecal streptococci | - | 20/100 ml |

A reasonable application of the effluent objectives would be to compare the average quality of runoff from a storm to the effluent criteria for a specified event such as the largest volume, or most polluted storm runoff in an average year.

The effluent guidelines set out by the Ministry are very stringent indeed and the need for a well-controlled management approach to runoff and possibly for treatment of effluent prior to discharge is evident.

Because receiving water modelling was not part of the terms of reference of this study, the runoff loading effect in relation to the receiving water criteria has not been assessed.

# 4 STORM WATER MANAGEMENT TECHNIQUES

## 4.1 General

Traditional philosophy on runoff of storm water from developed areas is undergoing some change in Canada and the United States, and in a number of European countries.

The traditional approach to urban drainage design is generally to attempt to provide for minimum flooding and inconvenience to urban dwellers, even under extremely adverse storm conditions. This approach obviously requires an efficient network to collect and transport flows away from developed areas and discharge them to receiving waters as quickly as possible. This results in large peaks and increased volumes of runoff to the receiving waters. The problems associated with this traditional approach to urban drainage design are now being recognized and some of the more critical are briefly summarized below:

1) The rapid wash-off of the ground surface results in discharge of contaminants to be assimilated by receiving waters.
2) Increased peak flows in downstream receiving waters often result in increased and severe erosion.
3) Increased imperviousness and rapid runoff decrease groundwater infiltration which can, over time, lower natural groundwater levels, affect vegetation, and decrease base flows in the natural watercourses.

Of prime concern in this study is point 1) above, that is, the pollutant load washed off the watershed and discharged to the Rideau River.

## 4.2 Storm Water Runoff Controls

A number of controls were considered and analysed to determine their ability to decrease loadings discharged from the watershed to the Rideau River. The types of controls which may be considered vary, depending on the land use and the extent of existing development. Controls can readily be incorporated into new developments but retroactive application is difficult and causes inconvenience to property owners in existing

developments. In addition, runoff controls which require surface ponding on large impervious areas, such as parking lots and rooftops, can be acceptable in certain industrial developments but severe opposition and criticism would be expected if they were applied in residential areas.

The runoff controls which were considered appropriate to the Merivale Area may be summarized under the following headings:

1) surface storage,
2) subsurface storage.

### 4.2.1 Surface storage

Surface storage includes those controls which were considered for containing runoff on the ground surface both at the source of the rainfall and throughout the system. Controls which result in surface storage at the source were considered only for undeveloped lands within the industrial areas since, as discussed previously, retroactive application of these controls to existing developments is not considered practical. Surface storage controls considered included:

a) rooftop storage,
b) roadway and parking lot storage,
c) surface storm water retention ponds.

Rooftop storage. Surface retention of storm water on large impervious roof areas in the catchment is one of the most economical and effective methods of retarding runoff from developed areas. This method of runoff control should generally be acceptable in industrial developments where large flat roof designs provide a substantial volume for storage of rainfall.

Roofs would be more than structurally adequate for storage of rainwater from the most severe storms anticipated. Present building code criteria for roofs in the Ottawa area require structural design to support snow loadings of 50 pounds per square foot, which is equivalent to approximately 9½-inches of water. This is more than satisfactory for the purposes described in this study and should not require additional expense, except perhaps some allowance to ensure watertightness of the roof.

Control devices for runoff from flat roofs are relatively simple and may consist simply of a steel ring of specific height, placed around the roof hoppers, with orifice holes in the ring at the lower levels. The orifices allow controlled discharge under most conditions but free overflow is permitted if the water level on the roof exceeds the height of the ring. A typical control device for roof hoppers is shown in Figure 3.

Alternatively, the owner of a building development may wish to direct the discharge from the roof to a separate storage tank with a controlled discharge to the sewer system. The storage tank would be equal to the specified storage for the equivalent roof area and may be located above or below ground. This method of control would reduce waterproofing problems and maintenance of large flat roofs, at some additional cost.

Parking lot storage. Many industrial developments, and particularly the type of development proposed for the Merivale Acres Industrial Colony, have large impervious parking lot areas associated with them. Surface retention of storm water on impervious parking lot areas is also an economical and effective method of retarding and reducing peak runoff. It is also considered to be very effective in settling solids and other associated pollutants out of the flow, which may then be mechanically removed from the ground surface after the storm.

Parking lots may be designed to store substantial volumes of the rainfall runoff from individual lots. Control structures in catch basins may consist simply of an orifice plate, placed below the inlet grate to restrict flows from the catchment area. Figure 4 shows a typical orifice plate control. Alternatively, a wall of specific height may be built around the catch basin with holes placed at the lower levels of the wall to act as orifice inlets to control rates of runoff to the sewer system. If the level of the water exceeded the elevation of the wall, free overflow conditions would result, reducing the possibility of severe property damage within the catchment area. Other methods of control, such as the Hydro-Brake* may also be considered.

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* Hydro-Brake flow regulator as designed and marketed by Hydro Storm Sewage Corporation of Olso, Norway, and New York, U.S.A.

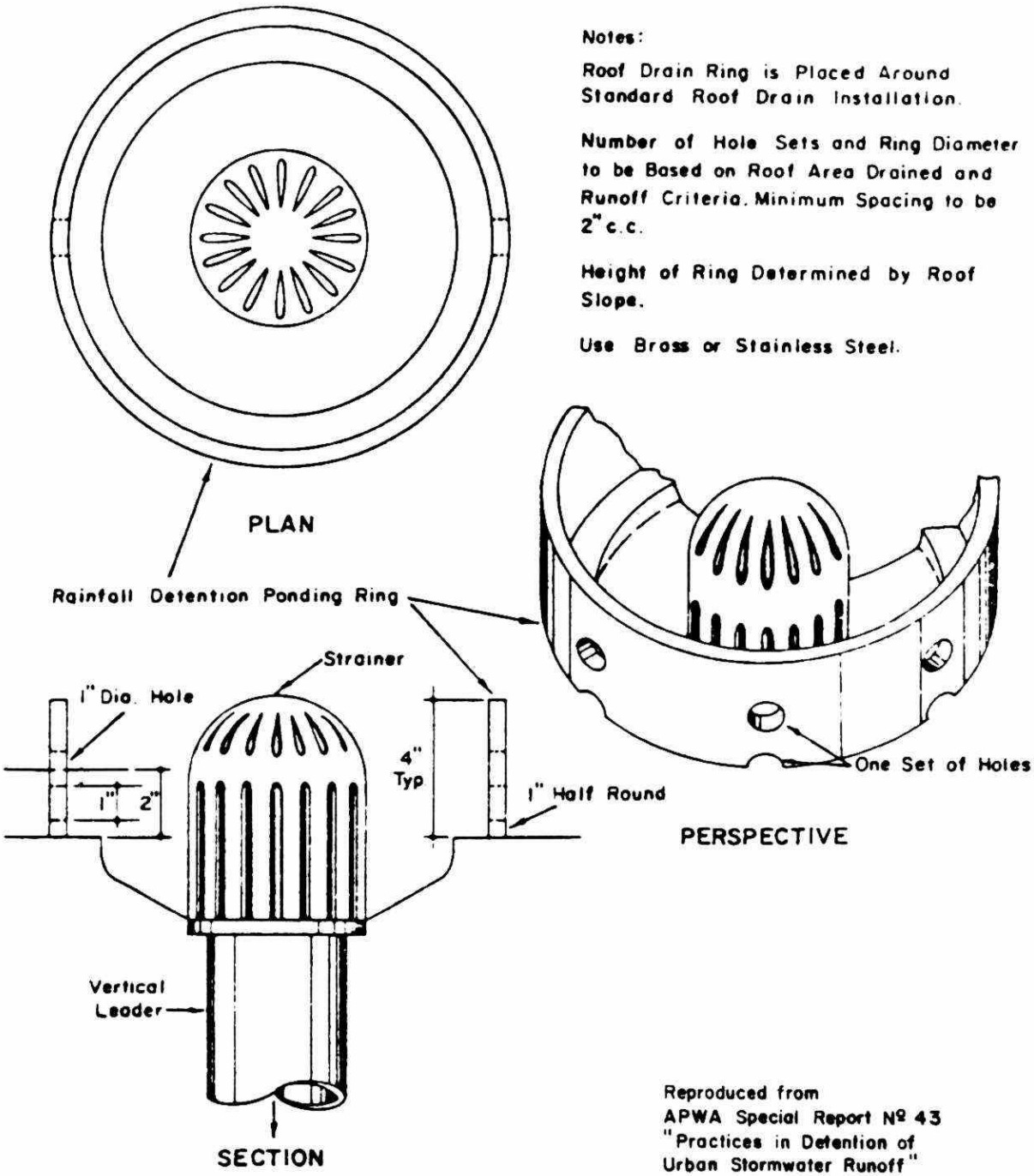
Notes:
Roof Drain Ring is Placed Around Standard Roof Drain Installation.
Number of Hole Sets and Ring Diameter to be Based on Roof Area Drained and Runoff Criteria. Minimum Spacing to be 2" c.c.
Height of Ring Determined by Roof Slope.
Use Brass or Stainless Steel.
PLAN
Rainfall Detention Ponding Ring
Strainer
1" Dia. Hole
1"
2"
4" Typ
1" Half Round
Vertical Leader
SECTION
One Set of Holes
PERSPECTIVE
Reproduced from
APWA Special Report Nº 43
"Practices in Detention of
Urban Stormwater Runoff"

FIGURE 3. RAINFALL DETENTION PONDING RING FOR FLAT ROOFS

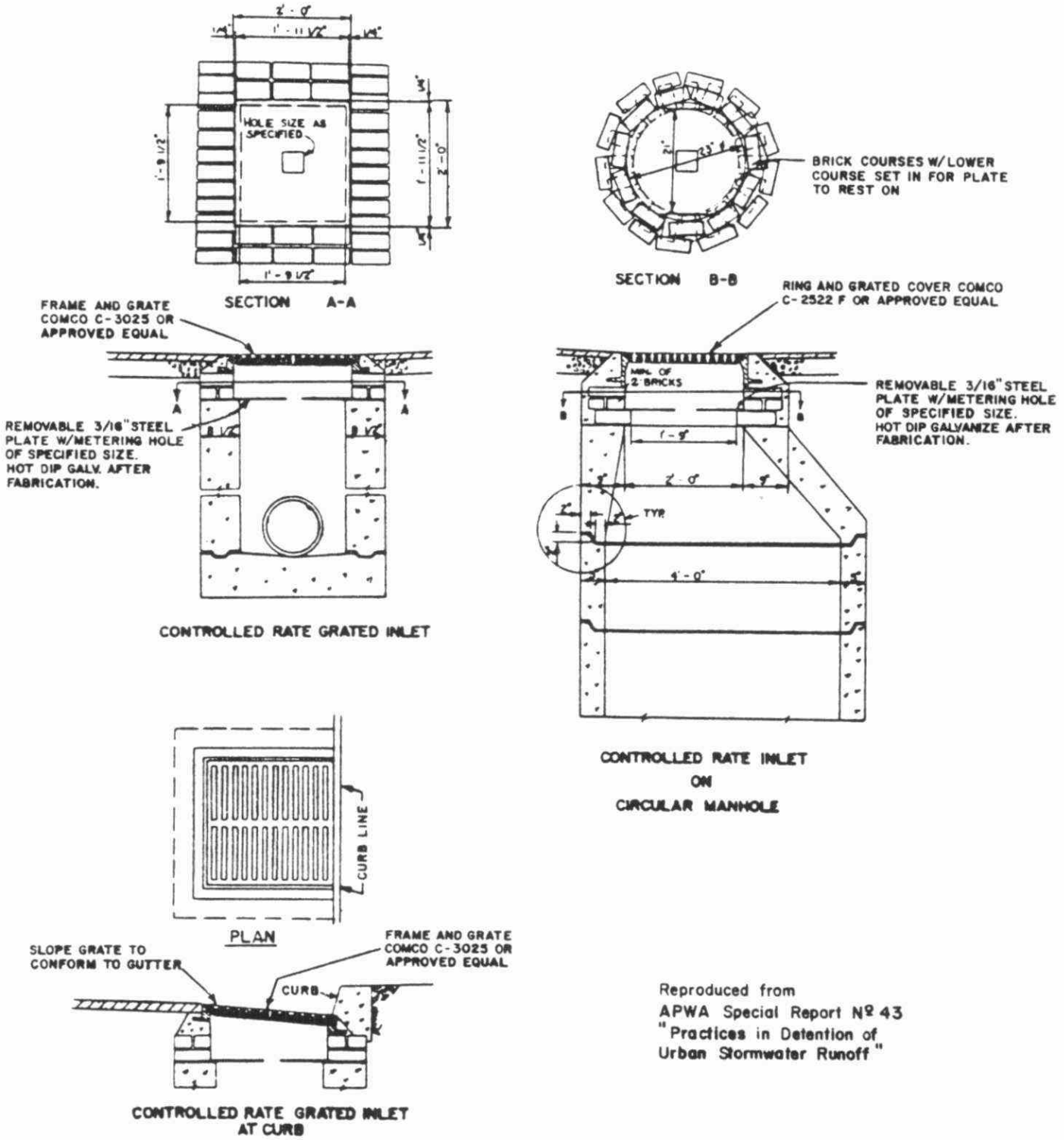
HOLE SIZE AS SPECIFIED
SECTION A-A
FRAME AND GRATE COMCO C-3025 OR APPROVED EQUAL
REMOVABLE 3/16" STEEL PLATE W/METERING HOLE OF SPECIFIED SIZE. HOT DIP GALV. AFTER FABRICATION.
CONTROLLED RATE GRATED INLET
BRICK COURSES W/LOWER COURSE SET IN FOR PLATE TO REST ON
SECTION B-B
RING AND GRATED COVER COMCO C-2522 F OR APPROVED EQUAL
REMOVABLE 3/16" STEEL PLATE W/METERING HOLE OF SPECIFIED SIZE. HOT DIP GALVANIZE AFTER FABRICATION.
CONTROLLED RATE INLET ON CIRCULAR MANHOLE
CURB LINE
PLAN
SLOPE GRATE TO CONFORM TO GUTTER
FRAME AND GRATE COMCO C-3025 OR APPROVED EQUAL
CURB
CONTROLLED RATE GRATED INLET AT CURB
Reproduced from
APWA Special Report Nº 43
"Practices in Detention of
Urban Stormwater Runoff"

FIGURE 4. CONTROLLED RATE INLETS

Extreme care must be taken in the design of the individual parking lots to minimize, as much as possible, inconvenience to drivers during heavy rains.

Surface storm water retention ponds. Storm water retention ponds have been operated very effectively in a number of municipalities across Canada, notably in Winnipeg, Manitoba. Such ponds are very effective for flow balancing to retard and reduce peak runoff. In addition, the retention pond will allow sedimentation of solids and associated pollutants and result in an improved quality of discharge. Studies presently underway are assessing the effectiveness of the ponds at Winnipeg for improving effluent quality.

Storm water retention ponds, if properly designed, can be made to blend aesthetically into the surrounding environment. Such facilities are particularly well suited to park areas, either within or adjacent to residential development. Ponding facilities built into park settings also provide additional year-round recreational benefits, such as boating and skating. The normal water quality in the pond, however, is expected to be such that swimming should not be allowed.

The National Capital Commission Greenbelt lands immediately opposite Bentley Avenue (Figure 1) are a possible location for a storm water retention pond within the Merivale area. This pond would collect the runoff from the Greenbelt lands, as well as from the Merivale Gardens residential subdivision, a total area of approximately 465 acres, or approximately 35% of the total drainage area.

A storm water retention pond must:

a) be effective in controlling the quality of discharge,
b) provide adequate protection from flooding in conjunction with the storm sewer system,
c) be a facility which is safe and desirable for recreational uses.

Experience and research have shown that the design of a storm water retention pond should include the following features:

1) Normal water depth in the pond should be about four feet as a minimum and six feet maximum. Storage requirements are

then provided over the normal water depth with additional freeboard above the design high water level.

2) Emergency overflow or bypass facilities must be provided if the water capacity of the system is exceeded by an extreme rainfall.
3) The pond should be sized to provide a minimum surface area of approximately five acres. This is aesthetically desirable and also will provide a facility large enough to be used for recreational activities.
4) Ponds should be oblong in shape and positioned with the long dimension in line with the prevailing winds. This will allow for maximum mixing of the water by wind action, to assist in the self-purifying process.
5) The pond banks should be no steeper than a ratio of 7 horizontal to 1 vertical for both safety and ease of maintenance. A ratio of 10 horizontal to 1 vertical is desirable where space permits.
6) Inlet and outlet pipes should be submerged and well marked, for safety reasons.

The process of sedimentation is such that the majority of solids and other associated pollutants are settled out of the flow in a sedimentation pond within the first few hours. Recent work on storm water has shown, however, that a detention time of 24 to 48 hours is required to attain suspended solids removal in the order of about 80 to 90 percent.* It has also been found that a detention time of 24 to 48 hours will generally result in very high coliform die-off.* This die-off is to be expected, as the bacteria are removed from their natural environment when placed in such a ponding facility.

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* Unpublished research on physical-chemical treatment of storm water by Ontario Ministry of the Environment under COA research project no. 76-8-37.

Unpublished research by Regional Municipality of Ottawa-Carleton under contract with the Central Mortage and Housing Corporation (CMHC) on Barrhaven storm water runoff treatment pilot study.

Unpublished research by the City of Winnipeg under contract with CMHC on treatment of urban runoff by storm water impoundment.

### 4.2.2 Subsurface storage

Subsurface storage includes those controls which provide impoundment and storage of storm runoff in underground or subsurface structures. For the Merivale area, considering the existing land use, the present scope of development and the development proposed, the following subsurface storage options were analysed:

a) pipe storage,

b) underground storage/treatment tank.

Pipe storage. The main trunk sewer serving the Merivale Industrial Area is 120-inch diameter from its outlet at the Rideau River westerly to Merivale Road, a distance of approximately 8,800 feet. The profile of the trunk sewer, as described above, is shown on Figure 5. Because of the grade or slope of the sewer, the available storage within the pipe is not its full volume but only a portion as shown on Figure 5. The total maximum available storage volume in the trunk sewer is calculated to be approximately 1.6 million imperial gallons.

To use this storage capacity, a control structure is located near the outlet from the sewer. The control structure is equipped with a weir to contain the runoff within the pipe. The weir length and elevation have been set to provide adequate capacity over the weir to match the flow capacity of the pipe. This provision has been made to control a designated storm runoff when the available pipe storage is already used.

The available storage is dependent on the elevation of the weir, and the maximum available storage is represented by the highest weir elevation. Although the higher weir elevation can be considered in assessing the presently serviced area, more outlet capacity will be required when the watershed becomes more fully developed and serviced with storm sewers. The weir will then have to be lowered, reducing the storage capacity to approximately 0.7 million imperial gallons.

A sluice gate equipped with a calibrated V-notch weir has been set in the weir wall to drain the storage volume at specific controlled rates. The sluice gate presently has a manual operator, but this may be mechanized in the future. The sluice gate mechanism could then be controlled by a water level sensor in the sewer.

Details of the control structure are shown on Figure 6.

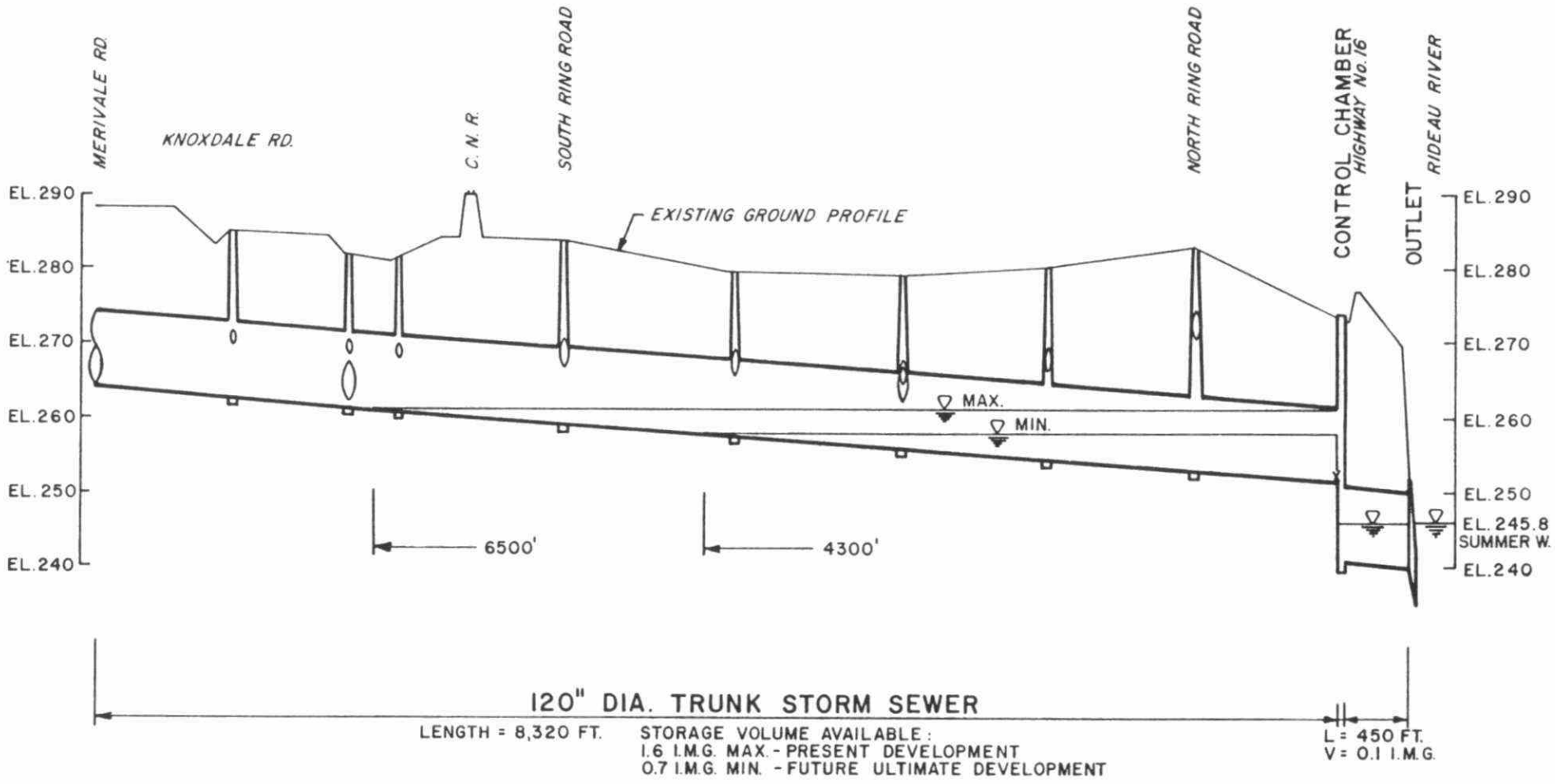


FIGURE 5. 120-INCH DIAMETER TRUNK STORM SEWER

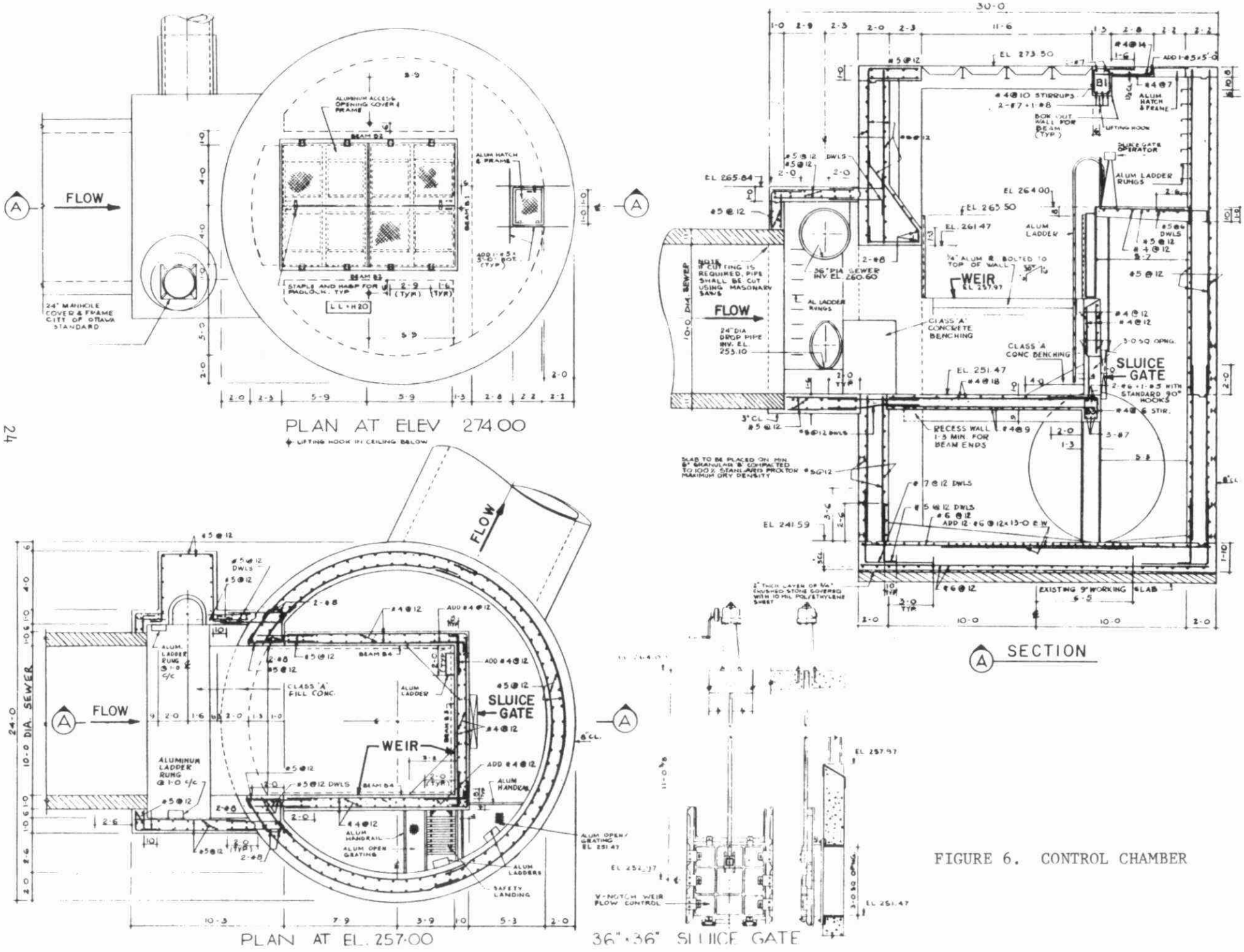


FIGURE 6. CONTROL CHAMBER

Underground storage/treatment tank. Underground storage may be provided either in the form of underground storage tanks with controlled outlets, or as a specially-designed very pervious granular sub-base to which the flow would be directed. Controlled drainage from the underground storage would be necessary, particularly during winter thaws and spring runoff, to ensure minimum damage to pavement surfaces by frost action.

The sewer system has been designed for the possible future requirement for an underground storm water retention/treatment tank. Figure 5 indicates that a drop of approximately nine feet has been provided in the pipeline at the control structure to allow a head loss through a future tank. It is expected that, if required, such a tank would provide storage and sedimentation of runoff prior to discharge to the Rideau River. Disinfection, if required would also be applied at this location.

The location proposed for the storage/treatment tank is shown on Figure 1 and is on land presently owned by the National Capital Commission. The NCC was formally notified, during previous easement negotiations, that a storage/treatment tank may be required at this location. No firm commitment, however, has been requested or received from the NCC with respect to the future use of this land.

## 4.3 Nepean Development Controls

The Township of Nepean, in the development of the Merivale Acres Industrial Colony, has advanced a planned approach to the management of storm runoff from new developments within the Merivale watershed. The storm water management controls applied to new developments by the Township are primarily source controls, as described previously herein.

The Township of Nepean has formulated a number of "Development Control Agreement Clauses" relating to the techniques proposed for the controlled management of runoff, which clauses are included in the site plan agreement and ultimately registered on the title. The development control agreement clauses are as follows:

## DEVELOPMENT CONTROL AGREEMENT CLAUSES

A. The Developer hereby agrees that the following shall be provided at the Developer's expense:

(1) The design of a storm water control system which will provide for a rate of runoff from the entire property (including rooftops) which shall be equal to or less than that which existed prior to development.

(2) The submission of the proposed design together with all supporting calculations shall be made to the Commissioner of Works for approval prior to the preparation of the Site Plan Agreement.

(3) The supply of all materials and the installation of the facilities.

(4) The repair and maintenance of the facility.

(5) To effectively control erosion of the site and to eliminate the transfer of solids to the storm sewer throughout the period of construction.

B. The Developer hereby agrees to the following terms:

(1) The Township of Nepean will be permitted lawful entry onto private property in order to:

(a) Clean up and remove settled solids from hard-surfaced areas where designated ponding of storm water has been established.

(b) Clean out and remove solids accumulation in the sumps of catch basins.

(c) Examine and adjust all storm water management devices.

(2) Not to alter grading, drainage or storm water management devices without prior approval of the Township of Nepean.

Design guidelines have also been prepared and are presented to the developer when he indicates an intention to develop land. The design guidelines are as follows:

## DESIGN GUIDELINES

### (1) DESIGN CRITERIA

(a) The layout and plan of the land to be developed shall provide a design of storm water flows and retentions in such a manner that the rate of runoff shall not exceed that which existed prior to development.

(b) The 5-year rainfall intensity curve shall be one of the elements applied in calculating runoff from the area to be developed.

### (2) LAND GRADING

(a) The land shall be graded and sloped in such a way that a minimum of surface runoff water shall be conducted to other property, including the public street.

(b) The grade or slope of the property shall be as gentle as possible in order to reduce erosion and the carrying of solids to the storm water catchment. Pavement areas, except where ponding is planned, shall be a maximum of 0.5% slope or as close thereto as practical. Grassed areas shall have a maximum slope of 1.0% or as close thereto as practical.

(c) Ponding areas shall be provided of a depth, size and extent which shall permit settling out of solids prior to flow into the storm sewer system. Design calculations shall be provided to support the extent of pond system and the time of retention.

(d) Pond depths of 2 feet, or more, shall be safeguarded by fencing, or other means, to reduce their hazardous aspects.

### (3) ROOF PONDING

(a) Ponding of water on rooftops shall be designed for a 4 inch depth. Automatic discharge devices shall be installed at all roof drains in order to gradually dispose of water to the storm sewer system.

(b) The responsibility of cleaning sediment that accumulates on the roof shall remain with the owner/lessee/renter.

(4) CATCH BASINS

(a) Standard catch basin frames, covers and barrels shall be used within paved areas.

(b) Orifice plates of 3/8 inch steel plate shall be installed at the lower level of the cast iron frame. Other control devices may be considered but the object being that the orifice shall be sized to permit only an acceptable calculated flow from the ponding area into the storm sewer system.

(c) Orifice plates or whatever other device is installed shall be capable of removal during winter periods, if this is found necessary.

(d) All catch basin barrels shall have 2 feet deep sumps in order to collect sediments.

(e) Catch basins installed in grass areas shall be of a ditch-receiver type which permits debris on the face of the grate to move up as the water depth increases.

# 5 ANALYSIS PROCEDURES AND RESULTS

## 5.1 Model Simulation

To adequately assess the relative benefits of the various storm water management controls considered and discussed in the previous section, it was necessary to apply modelling techniques. As discussed earlier, the Rational Method, which is the basic design tool for the calculation of runoff presently used by the Township, does not provide a hydrograph of flow and thus, the effects of storage either above ground or below ground cannot be adequately assessed. In addition, an estimate of pollutant loadings is of utmost importance to assessment of the effects of development on the Rideau River, and the effects of various storm water management controls on the pollutant washoff from the watershed. A modelling procedure which would estimate pollutant loadings under various conditions was required.

It was decided to apply the following models to assist in the analysis:

1) Storage, Treatment, Overflow, Runoff Model (STORM), as developed by the U.S. Army Corps of Engineers.
2) Storm Water Management Model (SWMM), as developed for the U.S. Environmental Protection Agency.

A more detailed description of each of these models and their functions as related to this study follows.

### 5.1.1 STORM

The STORM provides a means for analysis of the quantity and quality of runoff from urban or nonurban areas. The two main types of output are statistical information on quantity and quality of washoff and overflow, and pollutographs for selected individual events. Loads and concentrations of five basic water quality parameters are computed, including suspended and settleable solids, biochemical oxygen demand, total nitrogen, and orthophosphate. Land erosion is also computed.

The analysis provided by the model aids in sizing storage and treatment facilities to control the quantity and quality of storm runoff and land surface erosion. The model considers the interaction of seven storm water elements:

- rainfall/snowmelt,
- runoff,
- dry weather flow,
- pollutant accumulation and washoff,
- land surface erosion,
- treatment rates,
- detention reservoir storage.

STORM is a continuous simulation model, i.e., it is designed for a long period of record analysis using continuous hourly precipitation data. The output includes data for individual storms and the model is most useful in preparing an array of significant events on the basis of actual rainfall records.

5.1.2 SWMM

The SWMM is a mathematical model which will provide simulation of a specific storm and the related quantity and quality aspects of the storm runoff. This model is more sophisticated than the STORM and can analyse the effectiveness of various storm water management techniques for specific storm events.

The SWMM accepts any rainfall hyetograph or multiple hyetographs and produces a runoff hydrograph for the modelled watershed. Continuous runoff quality pollutographs are computed on the basis of the volume of the storm runoff and antecedent conditions, which include rainfall history, street sweeping data, land use and other related data.

Flow and quality routing through the transport system is accomplished by simulation of the physical system, allowing for variations in size, length, slope and cross-section configuration, as required.

Storage options may be internal (in-system) or external (located at the overflow point) and include the following:

a) surface storage,
b) in-line or system storage,
c) external storage reservoirs.

Treatment options may be used in conjunction with or independent of external storage. Treatment options include acceptable combinations of the following:

a) mechanically cleaned bar racks,

b) fine screens,

c) sedimentation,

d) dissolved air flotation,

e) microstrainers,

f) high rate filters,

g) effluent screens,

h) chlorination.

The receiving water block of the model simulates the effects of both quantity and quality of flow on the receiving body.

The structure of the Storm Water Management Model is shown in chart form on Figure 7, and the particular blocks of the model which were used in this study are indicated.

Parameters considered to be of major importance in preparing input data for both quality and quantity calculations in the SWMM include:

a) antecedent dry days,

b) street sweeping interval,

c) land use,

d) area imperviousness,

e) land slopes and surface retention,

f) infiltration,

g) design storm.

Where factual data was not available for the Merivale area, hypothetical parameters which appeared to be reasonable for application to this specific watershed were used.

Antecedent dry days. The antecedent dry days is defined as the number of days preceding a storm during which the total accumulated rainfall was less than one inch. This is very important parameter because it affects the quantity of pollutants which have accumulated on the ground surface to be washed off during the storm event.

An analysis of existing rainfall records indicated that a dry period (total accumulated rainfall less than one inch) of 30 days preceding a significant storm was, although severe, not unexpected.

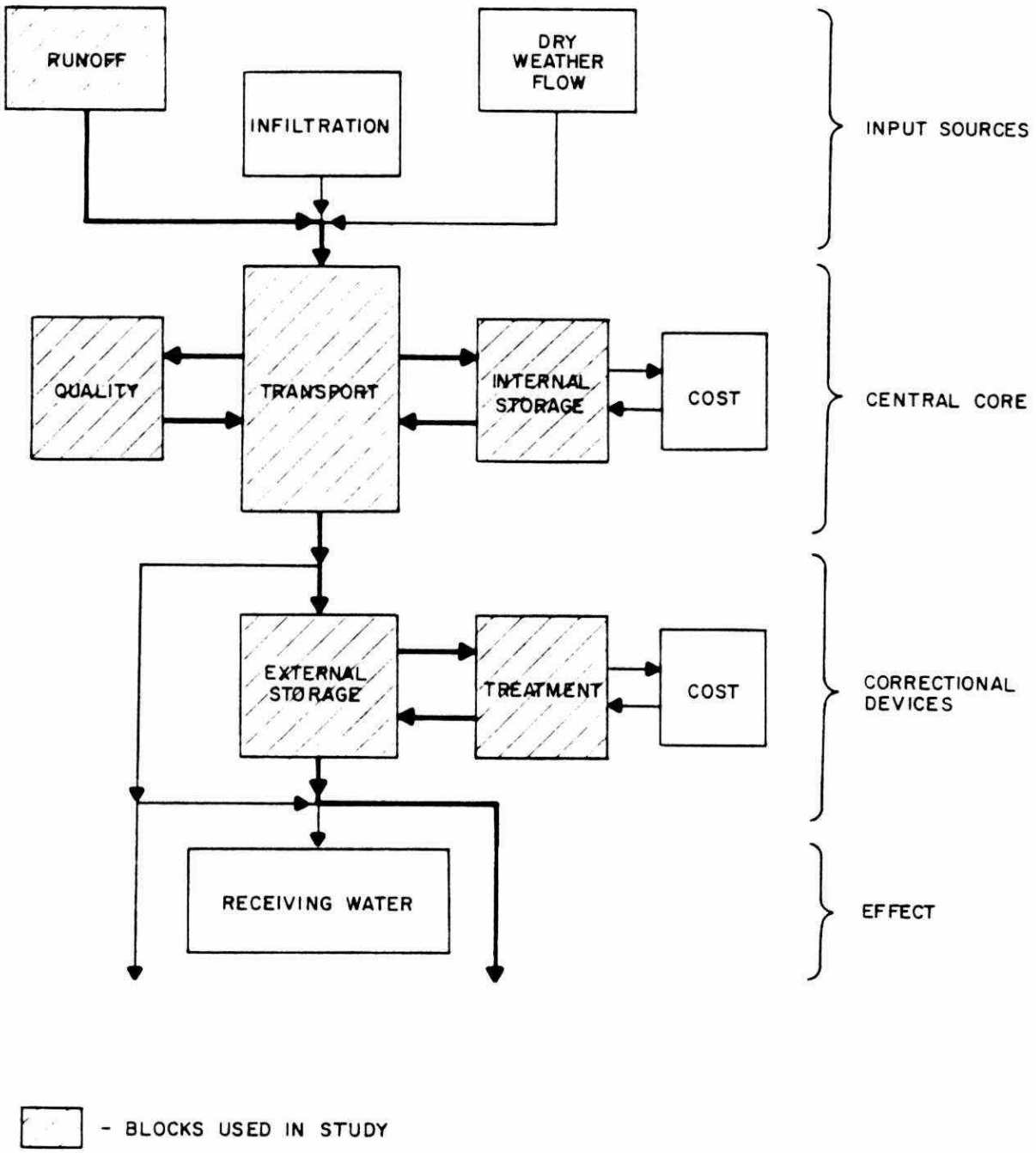


FIGURE 7. STORM WATER MANAGEMENT MODEL STRUCTURE

An antecedent dry period of 30 days was therefore used in this study for analysis of extreme events.

Street sweeping interval. The street sweeping interval is also an important parameter that affects quality of runoff. The model considers, as part of antecedent conditions, the interval since the streets were last mechanically swept.

It is understood that present normal practice is to undertake a complete street sweeping operation during the spring, after snowmelt and runoff conditions have subsided, and then on about two or three additional occasions throughout the summer. The model assumes an efficiency of 70% for normal mechanical sweepers.

To simulate actual conditions as closely as possible, a street sweeping interval of 60 days was assumed. This would eliminate the possibility of a street sweeping operation during the 30-day dry period. Two passes of the street sweeper was assumed to more realistically simulate the higher efficiency of the vacuum sweepers presently used by the Township of Nepean.

Land use. An important factor affecting the quality of runoff from the watershed is the land use distribution. There is a wide variety of land use within the Merivale watershed which varies from industrial to greenbelt forest. Land use designations used for the model simulation were taken from the Township of Nepean zoning designations and are shown on Figure 1.

Area imperviousness. Part of the input data to the Runoff block of the SWMM for the calculation of runoff hydrographs and pollutographs includes a factor which relates to the percent of the area that is impervious for each subcatchment. To determine realistic values for this factor, a detailed assessment was made of a number of areas within the watershed, which were considered to be representative of the overall area.

The percent imperviousness factors applied as input data to SWMM should not be confused with the runoff coefficient used in the Rational formula. The percent imperviousness factors describe the proportion of the area which is impervious and which contributes to direct and immediate runoff. The runoff coefficients used in the Rational formula

describe the percent of rainfall which comprises the runoff from a specific catchment area and are related to a number of characteristics of the area. Imperviousness is only one.

The following percent impervious factors were assumed for analysis. A comparison of these factors is made with the runoff coefficients of the Rational formula:

| Land Use Classification | SWMM percent imperviousness | Rational Formula runoff coefficient |
|---|---|---|
| Low Density Residential (Merivale Gardens) | 20% | 0.35 |
| Medium Density Residential (Tanglewood) | 35% | 0.50 |
| Medium Industrial | 40% | 0.60 |
| Greenbelt | 2% | 0.15 |

Land slopes and surface retention. The slope of the land is one factor that determines both rates of runoff from an individual catchment area and also the amount of water retained on the ground surface during the process of runoff. Because the Merivale area is generally very flat, a detailed analysis was undertaken to determine actual land slopes for each subcatchment. The base or default values suggested in the model were applied for surface retention for both pervious and impervious areas of the development.

Infiltration. Infiltration of rainwater into the ground is a major factor that influences the amount of runoff from a specific catchment area. Rates of infiltration are primarily determined by soil types, which have varying permeability coefficients.

With the assistance of the soils consultants, Golder Associates, a surface soils map was prepared outlining the major surface soil classifications. Infiltration rates were assigned to each subcatchment area on this basis.

Design storm. The Storm Water Management Model analyzes specific storms and an individual storm hyetograph is required as input

data. Keifer and Chu* developed a method for calculating a synthetic storm hyetograph from the rainfall intensity-duration curves normally used for design in the Rational Method. While the rainfall intensity-duration curves are created from a statistical review of a long series of rainfall records, the synthetic hyetograph transforms these average intensities from the rainfall intensity-duration curve at various times of concentration to one storm event that satisfies all average intensities for a specific storm duration. It is, therefore, a hyetograph enveloping all design storms used in the Rational Method. Statistical analysis of actual rainfall events generally shows the synthetic storm to have a return interval that is somewhat greater than the return interval of the corresponding rainfall intensity-duration curve.

For analysis and comparison of alternatives, the synthetic hyetographs derived from both the five-year and the one-year rainfall intensity-duration curves were developed using the method of Keifer and Chu and these are shown on Figure 8. An 80-minute storm duration was applied since this more than covers the time of travel from the headwater to the outlet of the main trunk sewer. This criteron is also in accordance with the method developed by Keifer and Chu.

The one-year rainfall intensity-duration curve was derived from the 2-year, 5-year, 10-year, 50-year and 100-year curves, using Gumbel probability paper.

## 5.2 Rainfall and Design Storm Analysis

A basis for hydraulic design of sewers has been established by most municipalities and, as discussed previously, the Township of Nepean has adopted the standard of protection provided on the basis of the five-year rainfall intensity-duration curve. The standard or design storm to be adopted for protection of receiving waters from pollution caused by urban runoff is, however, somewhat more complex. To assist in decision-making in this respect, an analysis was undertaken to compare the pollutant washoff from the watershed under various extreme conditions as follows:

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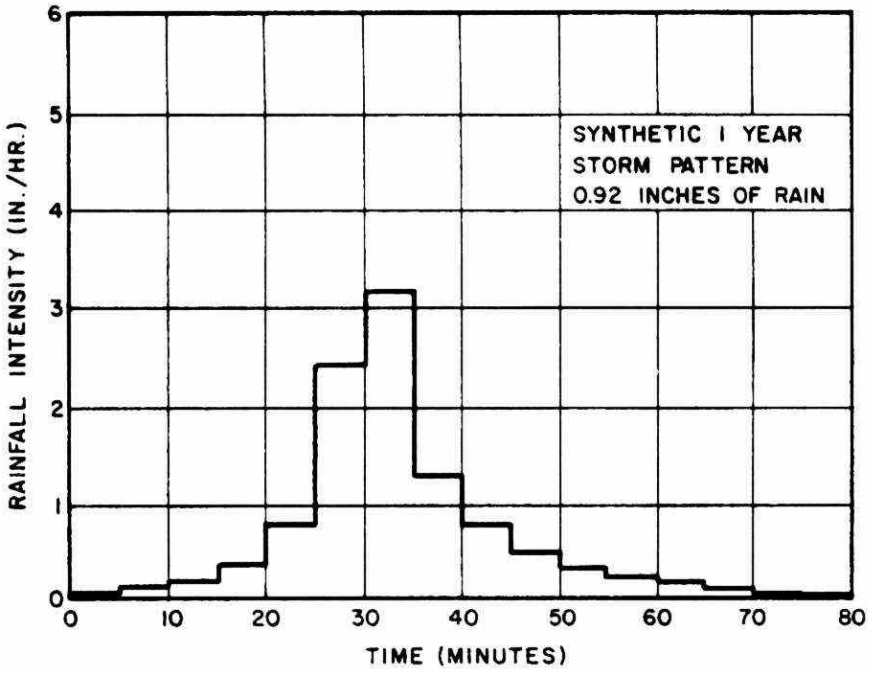
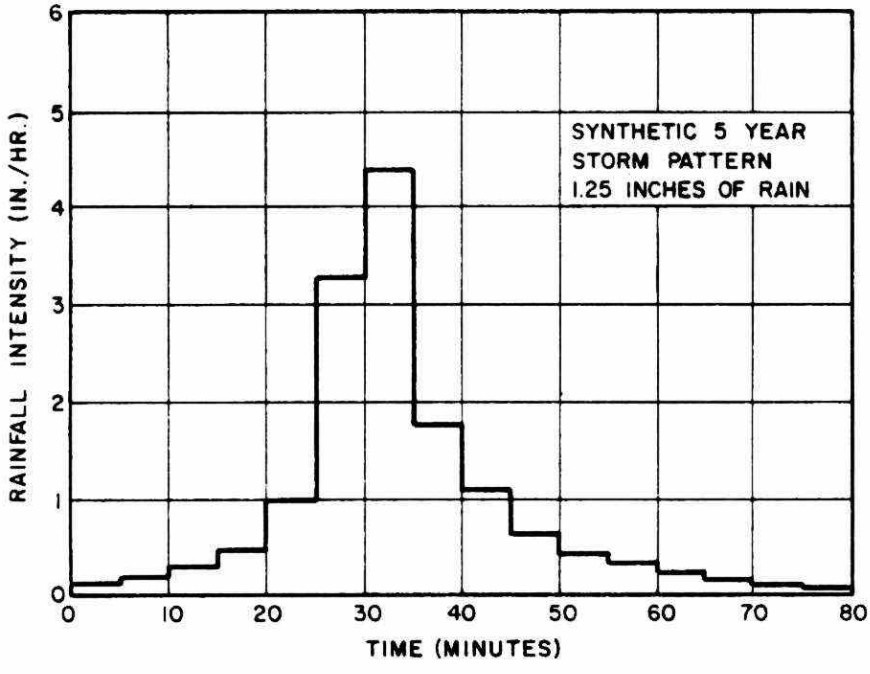
* Synthetic Storm Pattern for Drainage Design, by Clint J. Keifer and Henry Hsien Chu, Proceedings A.S.C.E., Paper 1332, August 1957.




REF:
"SYNTHETIC STORM PATTERN FOR DRAINAGE DESIGN"
C.J. KEIFER AND H.H. CHU
PROC. A.S.C.E. – AUG. 1957

BEFORE THE PEAK:

$$i = \frac{a\left[(1-b)\left(\frac{t_b}{r}\right)^b + c\right]}{\left[\left(\frac{t_b}{r}\right)^b + c\right]^2}$$




AFTER THE PEAK:

$$i = \frac{a\left[(1-b)\left(\frac{t_a}{1-r}\right)^b + c\right]}{\left[\left(\frac{t_a}{1-r}\right)^b + c\right]^2}$$

FIGURE 8. RAINFALL HYETOGRAPHS

a) SWMM simulation utilizing

   i) five-year synthetic hyetograph

   ii) 30 dry days preceding storm

   iii) no storm water management controls applied.

b) SWMM simulation utilizing

   i) one-year synthetic hyetograph

   ii) 30 dry days preceding storm

   iii) no storm water management controls applied.

c) STORM and SWMM simulation of actual rainfall events.

The total runoff and the peak rate of flow from the synthetic five-year storm, as expected, are substantially greater than those produced by the synthetic one-year storm. The hydrographs of flow, as simulated by the SWMM for the synthetic five-year and one-year storms, are shown on Figure 9.

A number of actual storms were modelled, using both SWMM and STORM to compare total washoff with the synthetic storms. Actual storms modelled and compared for this purpose all had relatively similar dry periods before the actual storm.

Table 2 summarizes the results of these initial model simulations.

TABLE 2. TOTAL LOADINGS OF POLLUTANTS IN WASHOFF

| Model Simulation | Storm Event | Duration (hrs) | Dry Days | Rain (in) | Runoff (in) | SS (lb) | $BOD_5$ (lb) | SS (mg/L) | $BOD_5$ (mg/L) |
|---|---|---|---|---|---|---|---|---|---|
| SWMM | 5 yr (syn) | 1.33 | 30 | 1.25 | 0.43 | 63,200 | 3,800 | 494 | 30 |
| SWMM | 1 yr (syn) | 1.33 | 30 | 0.92 | 0.22 | 41,570 | 2,570 | 640 | 40 |
| SWMM | 11/6/73 | 1.00 | 30 | 1.06 | 0.30 | 58,100 | 3,500 | 657 | 39 |
| STORM | 8/7/75 | 2.00 | 26 | 1.38 | 0.40 | 25,950 | 3,090 | 220 | 26 |
| STORM | 1/7/59 | 3.00 | 25 | 1.64 | 0.50 | 38,190 | 4,270 | 259 | 35 |

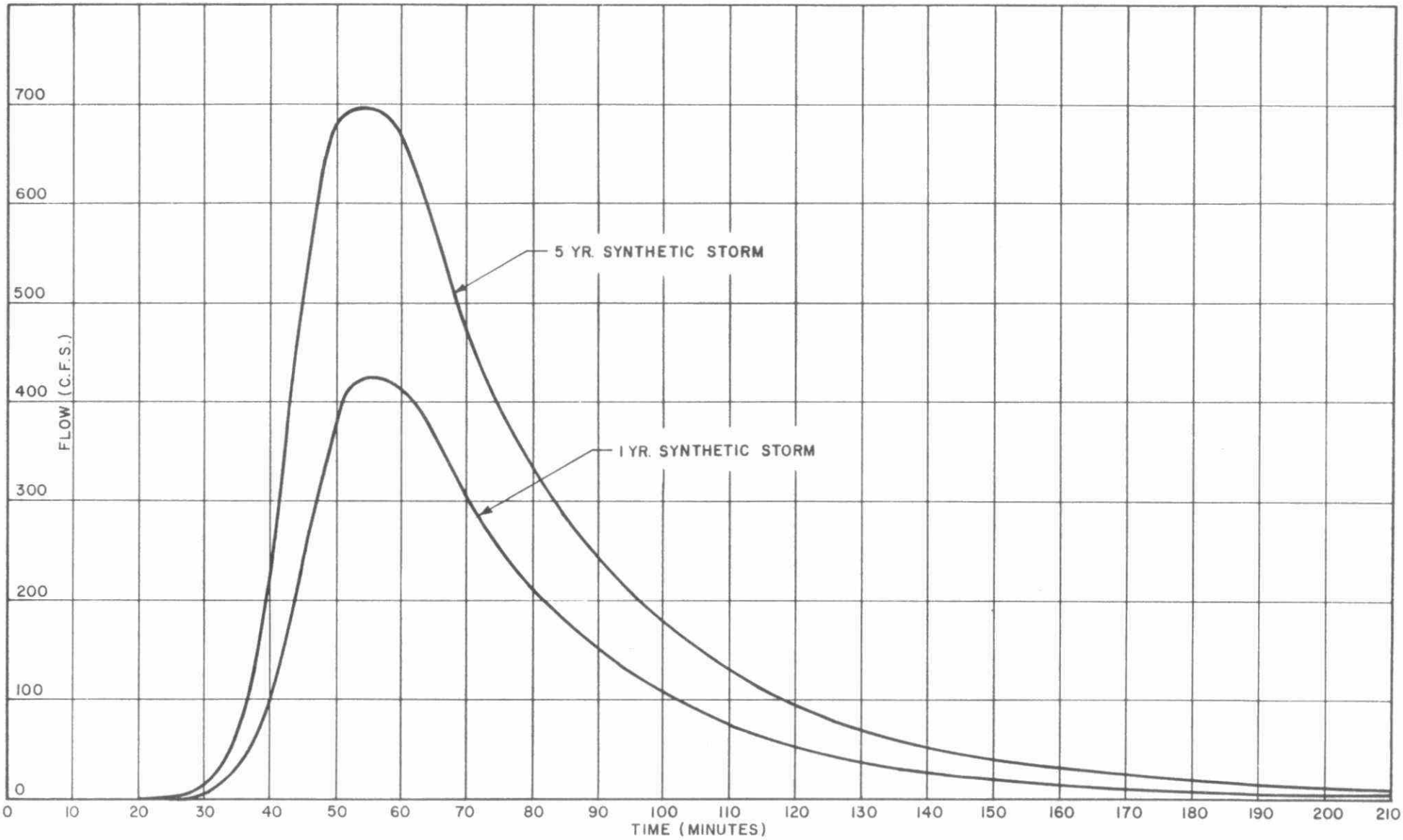



FIGURE 9. OUTFLOW HYDROGRAPHS WITHOUT RETENTION

Table 2 indicates that the total loading of pollutants in washoff, using SS and BOD as indicators, is very closely related to antecedent dry days as well as rainfall intensity. The analysis indicates that the majority of the accumulated pollutants will washoff the watershed under any intense storm condition. The one-year synthetic storm produced total washoff conditions that are of the same order of magnitude as the five-year synthetic storm and somewhat greater SS loadings than those predicted for actual storms of larger rainfall volumes but longer durations. Because of decreased volumes of runoff, the calculated pollutant concentrations from the one-year storm runoff were generally greater than for other storms analysed.

Based on the foregoing analysis of events, it was considered reasonable to use the one-year synthetic storm in the detailed evaluation of the effectiveness of various storm water control techniques.

## 5.3 Storm Water Management Surface Storage Controls

Storm water management surface storage controls which were considered appropriate to develop and analyse for the Merivale area were described in detail in Section 4 of this report under the following headings:

1) surface storage,
2) subsurface storage.

### 5.3.1 Surface storage options

Surface storage options considered, as described in Section 4, included:

a) surface retention on impervious areas of development including rooftops and paved parking areas,
b) storm water retention ponds.

Surface retention on impervious areas. The Storm Water Management Model is not presently capable of modelling the effects of storm water management controls, such as rooftop and parking lot storage, directly. To determine the effect of such controls, a detailed manual analysis was undertaken on a specific industrial development area within the Merivale Acres Industrial Colony, calculating rooftop and parking

lot storage volumes available and comparing these with runoff volumes for specific storm events. The industrial area studied was approximately 72 acres.

To assess the storage volumes available, some assumptions were required with respect to the type of development proposed, the total roof area available for storage and the total parking lot area available for storage. The planners' conceptual plan and the land uses permitted by Municipal By-Law No. 27-75, which controls the use of this area, were examined to enable realistic assumptions of the surface storage volumes available. It was assumed that buildings would cover approximately 15% of the gross area and would require an equal area of parking lot space, or approximately an additional 15% of the total gross area.

Assumptions were also required for the storage capacities of these areas and, in this respect, it was assumed that the rooftops would store the volume of rain which fell directly on them up to a predetermined maximum before free discharge. It was further assumed that the parking lots would store the runoff tributary to the parking lot drainage system, up to an average water depth of 4 inches over the parking lot areas.

Utilizing the surface storage available on rooftops and parking lots affects the rate of runoff from the area, although the total volume of runoff would be approximately the same. The analysis showed that peak rates of runoff could be reduced by 50 to 60%. A typical hydrograph showing the effect of surface storage controls for the analysis test area is shown on Figure 10.

It was necessary in the analysis of the overall effects of surface storage to modify input parameters to SWMM to account for these effects. Since surface retention on impervious areas was the control being analysed, it appeared appropriate to modify this input parameter to SWMM. Analysis showed that, for a typical industrial area (40% impervious), a surface retention of 0.7 inches average on impervious areas reduced peak rates of runoff from those areas by approximately 50%, thus simulating the effect of rooftop and parking lot storage. Therefore, for those areas where surface retention could be applied, a surface retention factor of 0.7 inches was used in the SWMM analysis.

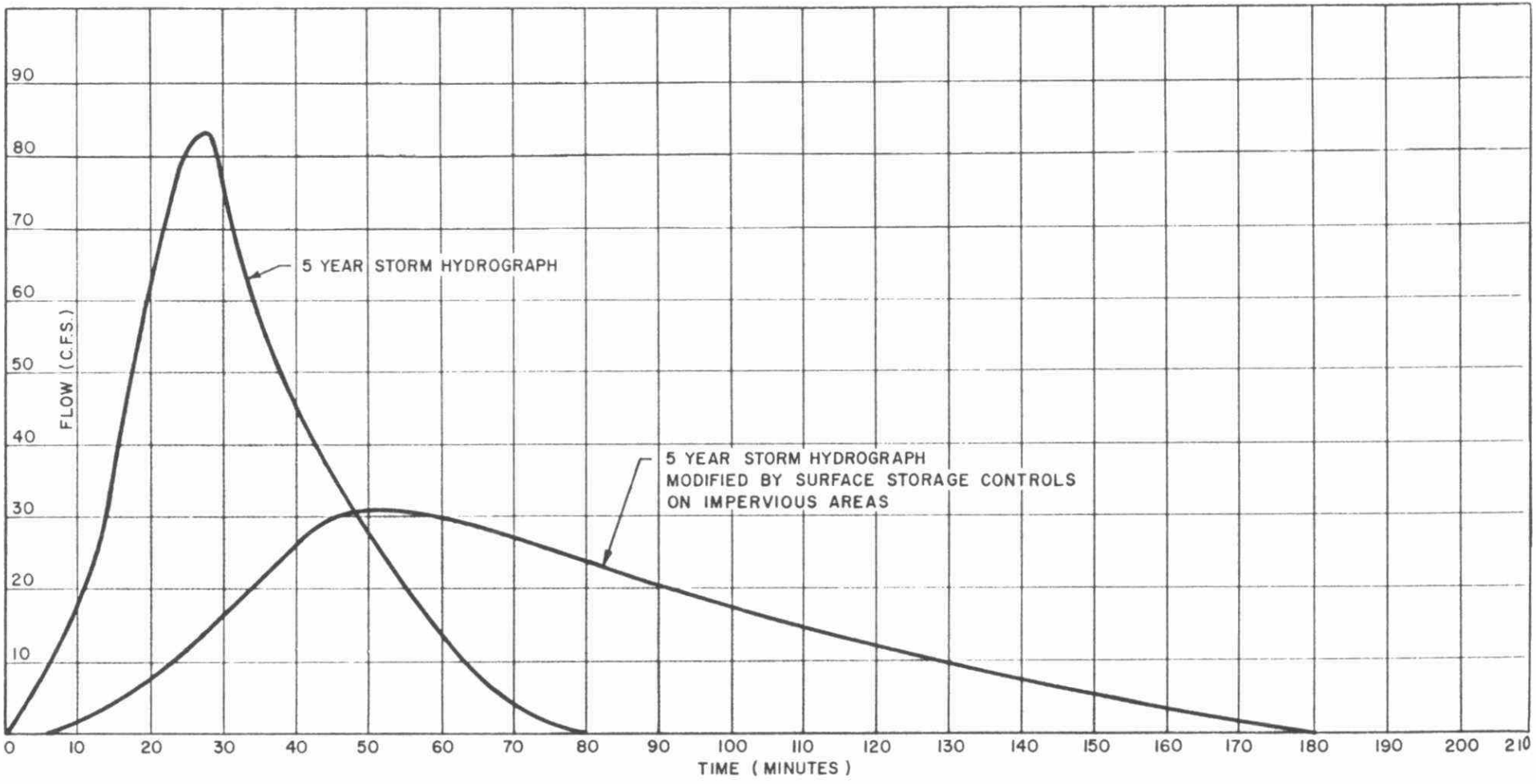



FIGURE 10. TYPICAL STORM DRAINAGE AREA (72 acres) HYDROGRAPHS

The SWMM assumes that the surface retention allowances do not enter the storm water collection and transport system and, as such, the output from SWMM does not entirely portray the surface retention control features. As discussed earlier, the surface retention controls allow outflow from these areas at predetermined controlled rates and, as such, the total volume of runoff is not significantly altered by the runoff controls. To account for this, flow hydrographs predicted by SWMM were modified to increase total volume of runoff to match the no-control situation.

The SWMM also assumes that pollutants associated with the increased surface retention do not re-enter the system. Although it is expected that a large portion of the pollutants will be settled out on the impervious surfaces, it is recognized that some portion of the pollutants will re-enter the system along with the controlled outflow. In this respect, it was assumed that approximately 50% of the suspended solids and 30% of the BOD associated with the surface retention waters could be settled on the impervious areas and removed by mechanical means after the storm event. Suspended solids and BOD pollutographs were, therefore, modified manually to account for this.

As discussed in Section 4, surface controls on impervious areas were applied only to proposed new development areas within the watershed. These included the Merivale Acres Industrial Colony, comprising approximately 200 acres, and the proposed new development area west of Merivale Road and north of Enterprise Avenue, comprising approximately 100 acres. These two new development areas, to which surface runoff controls were applied in the analysis, comprise a total area of approximately 300 acres, or approximately 23% of the total watershed.

Storm water retention pond. A number of locations were considered for storm water retention ponds. Of those considered, only the area within the NCC Greenbelt forest lands opposite Bentley Avenue appeared to be available as a desirable location for a surface storage retention pond. As noted in Section 4, a storm water retention pond in this area would collect the runoff from a total area of approximately 469 acres or approximately 36% of the total drainage area. Of this area, however, approximately 390 acres are greenbelt lands with generally lower associated pollutant contributions.

The approach to sizing the storm water retention pond was to utilize the STORM to analyse all rainfalls over a reasonably long period of record. The model then assessed, on the basis of rainfall, the runoff, the flow into storage, and the overflow in excess of preset treatment or flow-through rates, for various detention times. This analysis was undertaken for the watershed area tributary to the proposed pond. Curves showing predicted average annual overflow or bypass conditions for various sizes of retention facilities and for detention periods of 12 hours, 24 hours and 48 hours are illustrated on Figure 11.

Examination of Figure 11, shows that a flattening of the curves occurs at an average overflow or bypass condition of approximately three to five times per year. This flattening of the curves indicates that this particular storage size is a cost effective volume to provide. Additional storage volumes at additional cost would provide very little benefit in terms of treatment of flows.

On this basis, a storm water retention pond with a storage capacity of approximately 1.5 million imperial gallons was assumed for further analysis. Providing for a maximum rise in water level of about one foot, the area requirements under various conditions were as follows:

| | | |
|---|---|---|
| Gross area required | - | 8.5 acres |
| Water area at high water level | - | 7.0 acres |
| Water area at low water level | - | 5.5 acres |

### 5.3.2 Surface storage analysis results

Table 3 summarizes the quality of runoff from the entire watershed which resulted from analysis of the synthetic one-year storm event following 30 dry days, i.e. the total rainfall was less than one inch for 30 days preceding the storm.

Table 3 shows that, although the total runoff remained constant over the storm period, the total pounds of both SS and BOD were noticeably reduced even for the limited area to which storm water management controls were applied. Predictions generally show reductions in loadings of both BOD and SS in the order of 10 - 15% for the one-year storm event with maximum practicable controls applied to runoff to the system.

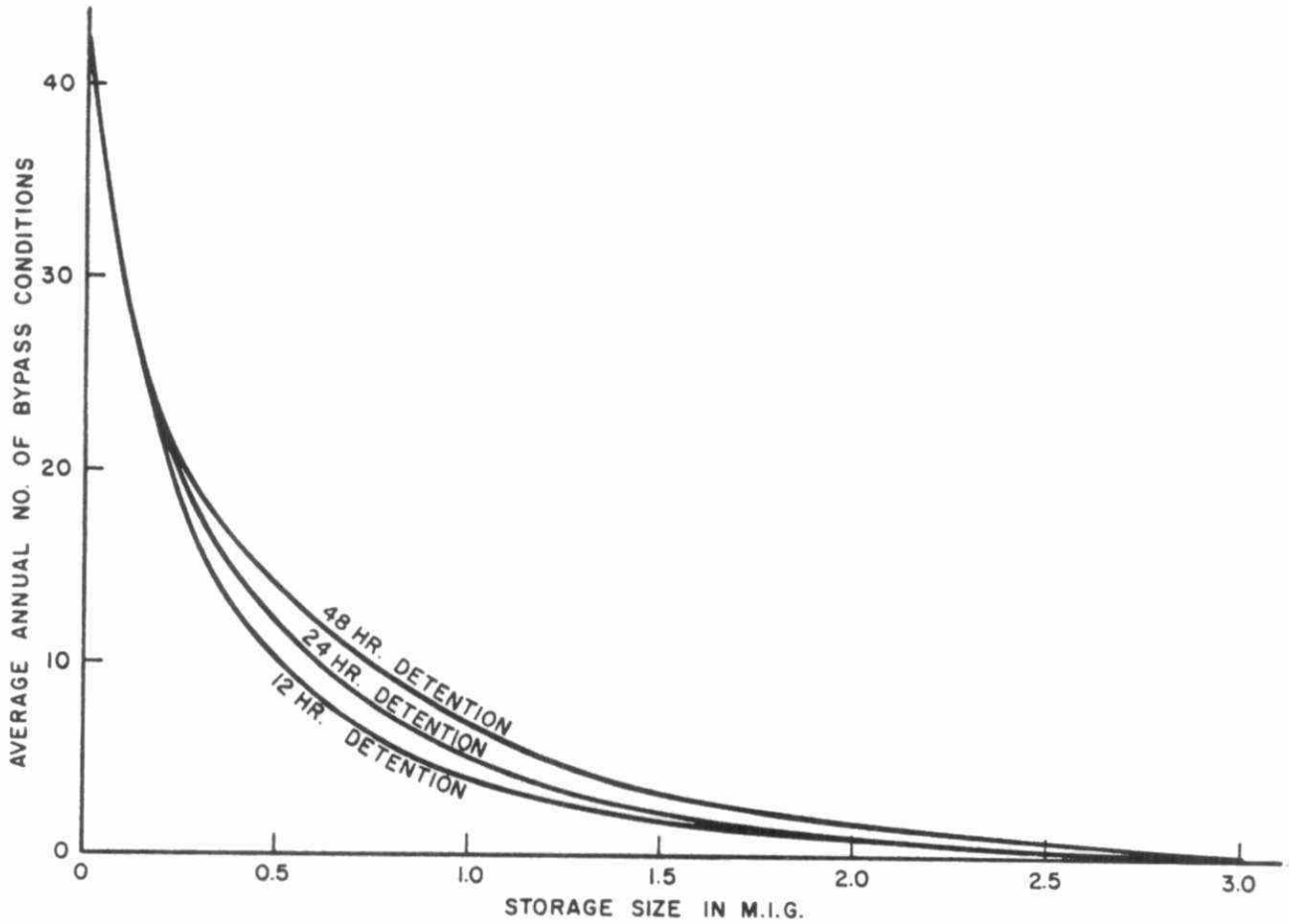



FIGURE 11. ANNUAL NUMBER OF BYPASS CONDITIONS
vs. PROPOSED RETENTION POND SIZE

TABLE 3. SYNTHETIC ONE-YEAR STORM SUMMARY

| SWMM Option | Total Runoff $ft^3$ | Total $BOD_5$ lb | Total SS lb | Avg. $BOD_5$ conc. mg/L | Avg. SS conc. mg/L |
|---|---|---|---|---|---|
| 1) No control | 1,037,560 | 2,570 | 41,570 | 40 | 640 |
| 2) Storage on Impervious areas of new development lands | 1,037,560 | 2,430 (95%) | 38,180 (92%) | 38 (95%) | 590 (92%) |
| 3) Pond on NCC Greenbelt land | 1,037,560 | 2,460 (95%) | 38,600 (93%) | 38 (95%) | 596 (93%) |
| 4) 2 + 3 | 1,037,560 | 2,310 (90%) | 35,200 (85%) | 36 (90%) | 544 (85%) |

The hydrographs and BOD and SS pollutographs predicted for the one-year synthetic storm event and the effects of the various storm water management controls analysed are shown on Figures 12, 13 and 14.

The analysis shows that, although the surface storage options do provide beneficial results with respect to pollutant reduction, predicted pollutant loadings to the receiving water would still far exceed the Ministry of the Environment guidelines for this particularly severe storm with the associated antecedent conditions assumed and for ultimate development of the watershed.

On the basis of this assessment, some additional treatment of the outflow from the system prior to discharge to the Rideau River would be required to meet MOE suggested guidelines.

## 5.4 Subsurface Storage/Treatment Controls

Subsurface storage options considered, as discussed in Section 4, included both:

a) pipe storage, and

b) an underground storage/treatment tank.

### 5.4.1 Storage size requirements

To properly assess the requirements for sizing subsurface storage, a statistical analysis of actual rainfall records applied to various sizes of storage facilities to predict average annual storage/treatment bypass

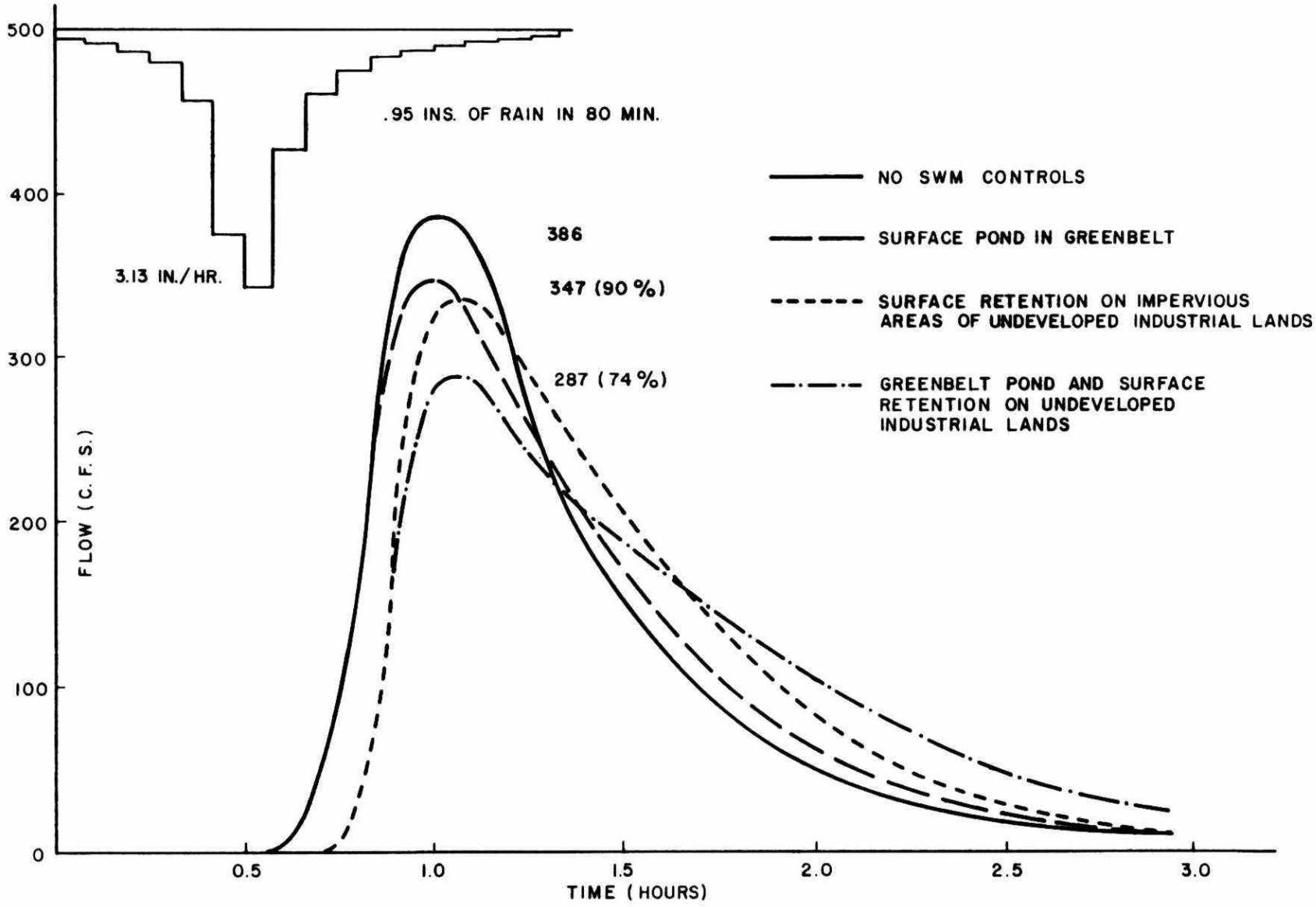



FIGURE 12. ONE-YEAR SYNTHETIC STORM HYDROGRAPHS

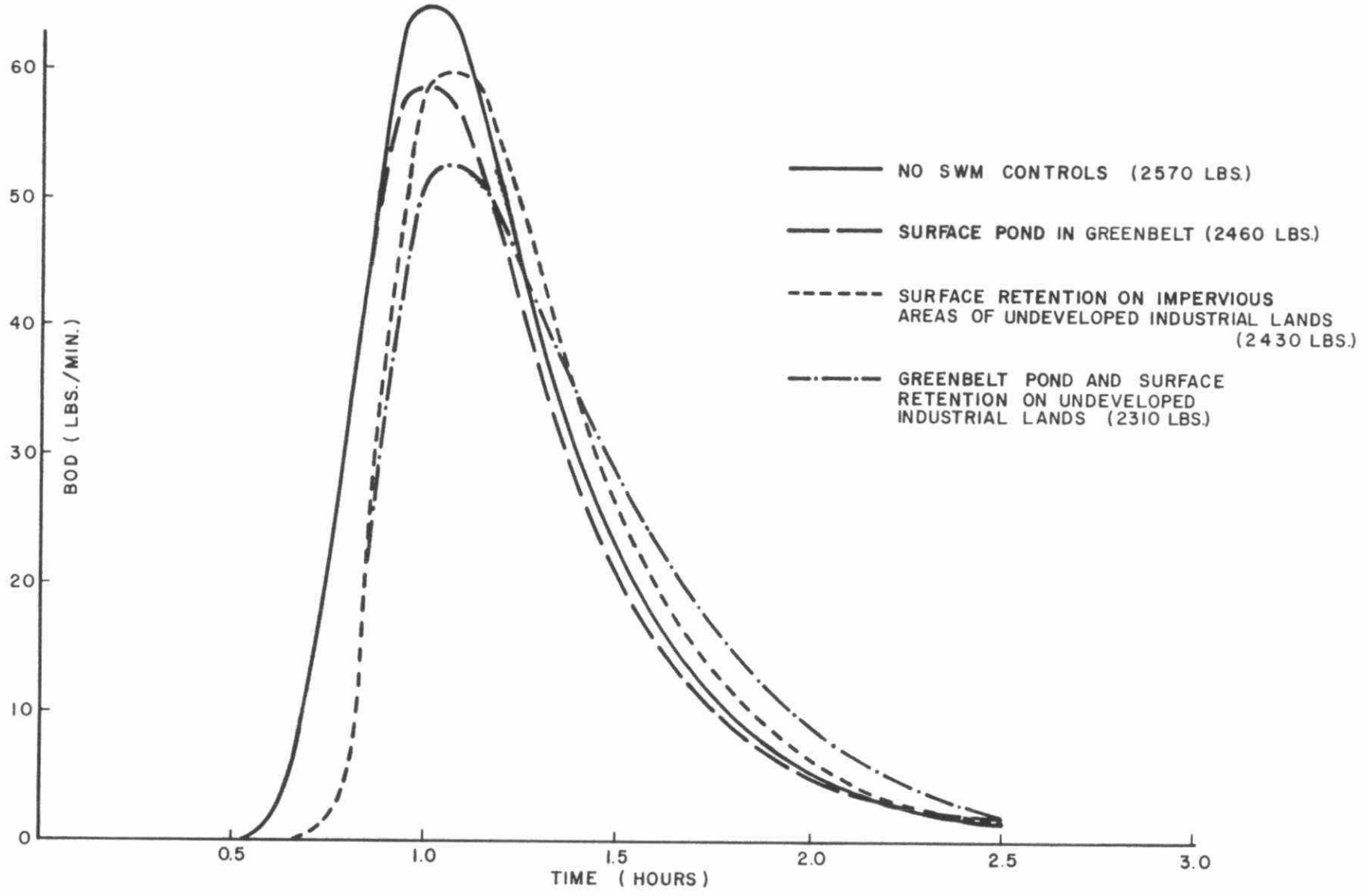



FIGURE 13. BOD POLLUTOGRAPHS

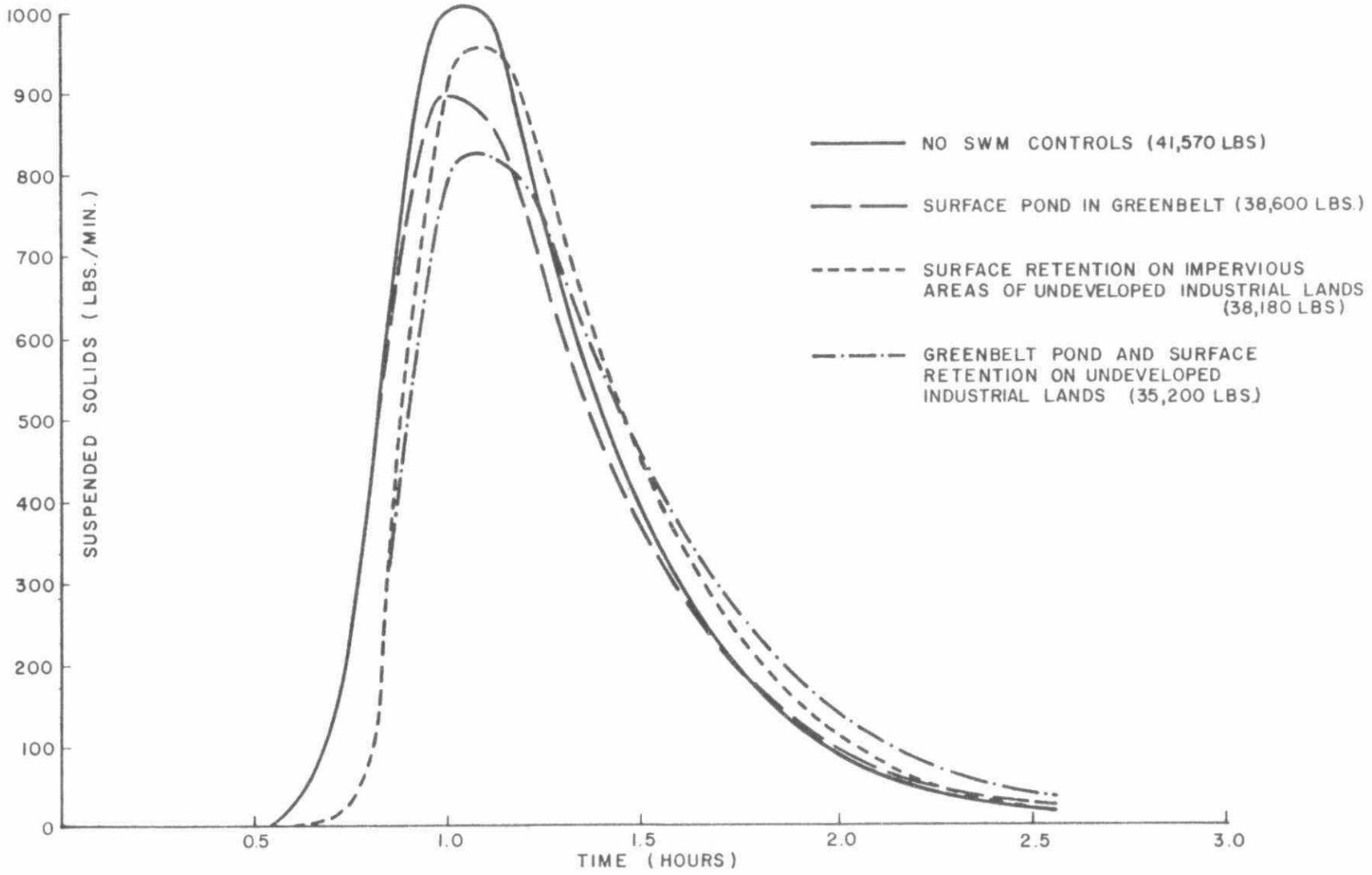



FIGURE 14. SUSPENDED SOLIDS POLLUTOGRAPHS

conditions was undertaken. The STORM was utilized to assist in the analysis of rainfall records using the same approach as described earlier for sizing the storm water retention pond.

In addition, various discharge rates from the storage/treatment facility corresponding to ranges of detention times considered reasonable for sedimentation of storm water were analysed. The upper or maximum treatment rate permitted corresponded to a detention time of approximately 1.5 hours and the lower or minimum rate permitted corresponded to a detention time of approximately 48 hours. The analysis was undertaken considering 18 continuous years of records from 1958 through 1975.

On this basis, probability curves for average annual bypass conditions were derived for various tank sizes and the different treatment rates discussed. The curves are shown on Figure 15. Examination of Figure 15 shows that, as expected, the predicted number of average annual bypass conditions is very much dependent on both size of storage facility and treatment rate or detention time within the storage facility.

Figure 15 shows that a storage facility of two million imperial gallons capacity with a detention time of three hours would result in an average of approximately six overflows per year. To provide a detention time of 48 hours, a storage facility of five million imperial gallons capacity would be required to limit the number of overflows to an average of about six per year.

Efficiency of sedimentation in the treatment of storm water is closely dependent on detention time. Although little research data is available on this aspect, preliminary research by the Ontario Ministry of the Environment suggests that:

> "Natural sedimentation for up to 48 hours has produced pollutant reductions in the order of 92% SS, 70% BOD and 60% COD."*

Further analysis using alum as a coagulant at a dosage of about 30 mg/L to assist sedimentation showed that:

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* Unpublished research on Physical-Chemical Treatment of Storm Water, by Ontario Ministry of the Environment under COA project no. 76-8-37.

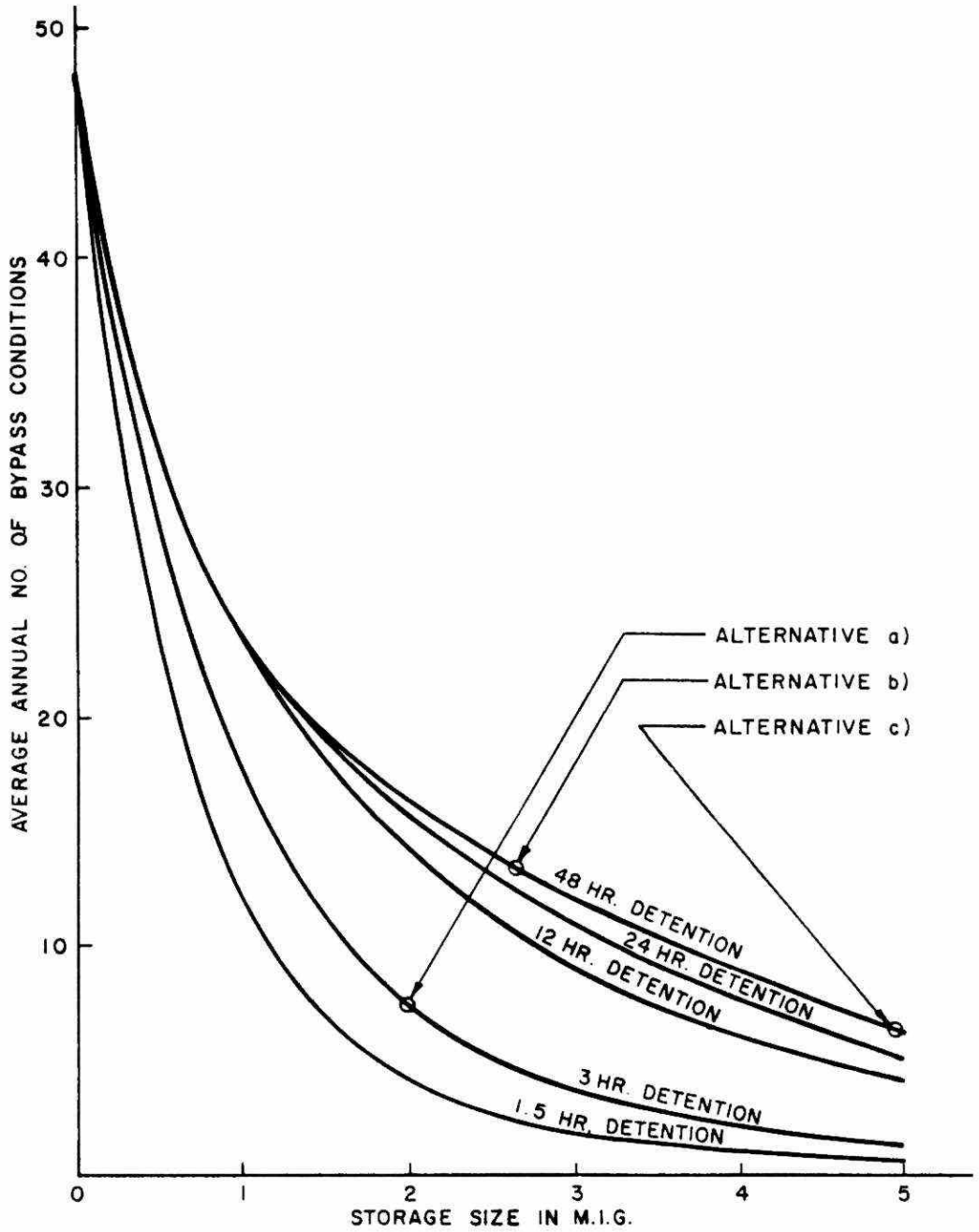


FIGURE 15. ANNUAL NUMBER OF BYPASS CONDITIONS vs. PROPOSED STORAGE SIZE

"Pollutant reductions under these conditions, including two hours settling, have been in the order of 98% SS, 72% BOD, 87% COD and 80% soluble P."*

The MOE also reports that:

"Fecal reductions have been in the order of 96% and 87% for chemically assisted and natural sedimentation, respectively."*

Considering, therefore, the relationship of storage volume to detention time and effectiveness in treatment, it was felt to be desirable to consider and compare the following storage/treatment alternatives:

a) A two million imperial gallon storage facility with chemically-assisted sedimentation and a detention time of approximately three hours. From Figure 15 it is noted that this operation would result in approximately six bypass conditions per year.

b) A two million imperial gallon storage facility, combined with approximately 0.7 million imperial gallons available pipe storage, with natural sedimentation and a detention time of approximately 48 hours. Figure 15 shows that this operation would result in approximately 13 bypass conditions per year.

c) A five million imperial gallon storage facility with natural sedimentation and a detention time of approximately 48 hours. Figure 15 shows that this operation would result in approximately six bypass conditions per year.

### 5.4.2 Storage/treatment analysis results

The operation of the proposed storage/treatment facility was analysed for two basic conditions:

1) average annual conditions,
2) single-event operation.

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* Unpublished research on Physical-Chemical Treatment of Storm Water, by Ontario Ministry of the Environment under COA project no. 76-8-37.

Average annual conditions. Utilizing the STORM, average annual runoff and loading conditions may be predicted on the basis of actual rainfall history. Table 4 summarizes predicted flow and associated average water quality runoff from the watershed.

TABLE 4. SUMMARY OF AVERAGE ANNUAL WASHOFF FROM WATERSHED

| Flow | SS | BOD | P |
|---|---|---|---|
| 20,480,000 $ft^3$ | 115,100 lb | 17,440 lb | 625 lb |
| | 90 mg/L | 14 mg/L | 0.5 mg/L |

On an average annual basis, it was predicted that approximately 20.48 million cubic feet of flow would be washed off the watershed. Approximately 115,000 lb of SS and 17,890 lb of BOD would be associated with this flow, resulting in average concentrations of approximately 90 mg/L SS and 14 mg/L BOD. These loadings also may be translated into loadings of approximately 88 lb/acre per year for SS and 13 lb/acre per year for BOD from the entire watershed. This assessment was made utilizing the STORM and assuming that the watershed was fully developed and serviced in accordance with the land use designations and storm sewer system outlined on Figure 1. The STORM is not capable of assessing the relative merits of surface storm water management controls and, therefore, the effects of these features were not included in the above analysis. On the basis of the previous analysis, properly implemented and managed runoff controls could reduce the predicted loadings from the watershed by approximately 10%-15% for BOD and SS.

The STORM does not model treatment. However, STORM will, for various treatment rates, predict the proportion of the total washoff from the watershed directed into treatment, and the proportion which will be bypassed. Using this analysis, the various storage/treatment alternatives discussed previously, along with treatment efficiencies considered appropriate and summarized below, were considered:

a) - a two million imperial gallon storage/treatment facility,
   - chemically-assisted sedimentation,
   - three-hour detention time,
   - treatment efficiency (for flow intercepted),

SS removal 90%
BOD removal 70%
P removal 50%.

b) - a two million imperial gallon storage/treatment facility + 0.7 million imperial gallons sewer storage,
- natural sedimentation,
- 48-hour detention time,
- treatment efficiency (for flow intercepted),
SS removal 90%
BOD removal 70%
P removal 50%.

c) - a five million imperial gallon storage/treatment facility,
- natural sedimentation,
- 48-hour detention time,
- treatment efficiency (for flow intercepted),
SS removal 90%
BOD removal 70%
P removal 50%.

Table 5 summarizes predicted average annual flow and quality of runoff to the Rideau River, considering managed runoff from the development and comparing the various treatment alternatives described earlier.

Examination of Table 5 shows that alternative a), which is the provision of a two million imperial gallon storage/treatment tank, along with chemical coagulant addition to assist sedimentation, was predicted to provide a quality of effluent which, under average annual conditions, is essentially within the limits of the guidelines suggested for discharge to the Rideau River. Alternative c) was also predicted to provide a quality of discharge under average annual conditions which is essentially within the limits of the MOE guidelines. However, this alternative requires a five million imperial gallon storage/treatment facility, compared to the two million imperial gallon facility required under alternative a), at approximately double the capital cost. A portion of the five million imperial gallon storage requirement may be provided

TABLE 5. SUMMARY OF AVERAGE ANNUAL FLOW AND QUALITY OF RUNOFF CONSIDERING MANAGED RUNOFF AND TREATMENT

| | ALTERNATIVE a)<br>- 2 MIG tank<br>- Chemically-assisted sedimentation<br>- 3 hr detention | | | | ALTERNATIVE b)<br>- 2 MIG Tank + 0.7 MIG sewer storage<br>- Natural sedimentation<br>- 48 hr detention | | | | ALTERNATIVE c)<br>- 5 MIG storage (tank + sewer)<br>- Natural sedimentation<br>- 48 hr detention | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Flow ($ft^3$) | SS (lb) | BOD (lb) | P (lb) | Flow ($ft^3$) | SS (lb) | BOD (lb) | P (lb) | Flow ($ft^3$) | SS (lb) | BOD (lb) | P (lb) |
| 1) Washoff from Watershed (modified to account for Storm Water Management controls)* | 20,480,000 | 97,800 | 15,700 | 560 | 20,480,000 | 97,800 | 15,700 | 560 | 20,480,000 | 97,800 | 15,700 | 560 |
| 2) Flow Into Storage/ Treatment | 17,010,000 | 78,900 | 13,400 | 460 | 12,580,000 | 58,300 | 9,900 | 340 | 16,310,000 | 75,600 | 12,800 | 440 |
| 3) Flow Out of Storage/ Treatment** | 17,010,000 | 7,890 | 4,000 | 230 | 12,580,000 | 5,830 | 2,970 | 170 | 16,310,000 | 7,560 | 3,840 | 220 |
| 4) Bypass Around Storage/Treatment | 3,470,000 | 18,900 | 2,300 | 100 | 7,900,000 | 39,500 | 5,800 | 220 | 4,170,000 | 22,200 | 2,900 | 120 |
| 5) Total Loading to Rideau River | 20,480,000 | 26,790 (21 mg/L) | 6,300 (4.9 mg/L) | 330 (.25 mg/L) | 20,480,000 | 45,330 (35.5 mg/L) | 8,770 (6.9 mg/L) | 390 (.31 mg/L) | 20,480,000 | 29,760 (23 mg/L) | 6,740 (5.2 mg/L) | 340 (.26 mg/L) |

* Loadings from Table 4 reduced to account for source storm water management controls, SS reduced 15%, BOD and P reduced 10%.

** Treatment efficiencies: SS 90% removal, BOD 70% removal, P 50% removal.

within the trunk sewer, which would reduce the tank size proportionally. Approximately 0.7 million imperial gallons could be provided within the sewer, with the remaining 4.3 million imperial gallons provided in the form of an underground tank.

Alternative b) essentially involves the use of the storage tank volume provided under alternative a), along with the available storage within the trunk storm sewer and the provision of a 48-hour detention time. This approach would obviously result in more overflow/bypass conditions on an annual basis and, therefore, the total loading to the Rideau River is predicted to be somewhat greater than for alternatives a) or c). However, this method is also predicted to produce a quality of discharge which approaches very closely the limits of the guidelines suggested by the MOE and may be acceptable.

After consideration of the above analysis of alternatives, it was concluded that the most desirable approach was that described as alternative b), which involves the utilization of the available trunk sewer volume for storage along with a two million imperial gallon tank. The tank would be required when increased development and extension of the storm sewer services within the area require the sewer volume for flow capacity. The storage volume should be operated to provide approximately 48 hours detention of the flow prior to discharge.

If continuous monitoring shows a quality of discharge which is unacceptable, the addition of alum as a coagulant to assist sedimentation should be considered.

Earlier discussion in this report described how the antecedent dry conditions affected the total loading of pollutants washed off the watershed for specific storms. Analysis of actual rainfall records using the STORM allows arraying of significant events to compare this effect. Table 6 shows an array of the major storms which produced bypass conditions at the storage/treatment tank over a period of 18 years from 1958 to 1975. The 12 most severe storms are shown in order of magnitude, based on rainfall and runoff.

It is noteworthy that the most severe storms do not necessarily produce the most severe pollution loading conditions. The largest rainfall analysed occurred on September 12, 1963. This storm produced a

TABLE 6. ARRAY OF SIGNIFICANT STORMS

| | Duration (h) | Dry Days | Rain (in) | Runoff (in) | SS (lb) | BOD (lb) | SS (mg/L) | BOD (mg/L) |
|---|---|---|---|---|---|---|---|---|
| 12/9/63 | 9 | 8 | 2.16 | 0.69 | 5,146 | 696 | 44 | 8 |
| 7/8/72 | 7 | 3 | 1.89 | 0.62 | 3,265 | 368 | 35 | 4 |
| 13/7/72 | 5 | 0 | 1.73 | 0.51 | 646 | 91 | 6 | 1 |
| 21/9/67 | 6 | 17 | 1.71 | 0.55 | 11,892 | 1,587 | 133 | 18 |
| 1/7/59 | 3 | 25 | 1.64 | 0.50 | 38,193 | 4,266 | 404 | 45 |
| 12/7/72 | 2 | 1 | 1.51 | 0.45 | 8,952 | 1,039 | 70 | 8 |
| 20/7/75 | 3 | 11 | 1.49 | 0.45 | 5,168 | 726 | 62 | 9 |
| 8/8/68 | 6 | 17 | 1.47 | 0.48 | 13,031 | 1,495 | 175 | 20 |
| 29/6/70 | 2 | 1 | 1.44 | 0.43 | 1,239 | 171 | 10 | 1 |
| 8/8/73 | 3 | 6 | 1.40 | 0.46 | 10,243 | 1,121 | 104 | 12 |
| 8/7/75 | 2 | 26 | 1.38 | 0.40 | 25,954 | 3,089 | 222 | 26 |
| 12/7/62 | 3 | 18 | 1.19 | 0.35 | 10,199 | 1,350 | 143 | 36 |

rainfall of 2.16 inches over nine hours and an associated runoff of 0.69 inches. The BOD loading associated with this storm was calculated by the model to be 696 lb and the SS loading was calculated to be 5146 lb, for concentrations of 8 mg/L and 44 mg/L for BOD and SS, respectively.

The largest BOD and SS loadings were predicted to be produced by a storm of July 1, 1959, which resulted in a rainfall of 1.64 inches over three hours. The runoff was calculated to be 0.50 inches. The BOD loading from this storm was calculated to be 4266 lb and the SS loading 38,193 lb, for concentrations of about 45 mg/L and 404 mg/L for BOD and SS, respectively.

Table 6 also shows the dry days preceding each storm. The number of dry days preceding the largest rainfall (1963) was only eight, whereas 25 dry days preceded the lesser volume storm (1959). The effect of build-up of pollutants on the ground surface over a dry period is clearly shown, indicating the desirability of an effective street sweeping policy.

Single event operation. To analyse the operation and effectiveness of the two million imperial gallon storage/treatment facility for a single relatively severe event, the External Storage/Treatment block of the Storm Water Management Model was used. As with the analysis of surface storm water management controls, a synthetic one-year storm following a 30 dry-day period was analysed.

As noted previously, manual modifications were made to both hydrographs and pollutographs predicted by the SWMM to more realistically represent the predicted effectiveness of the surface storm water management control features.

The present SWMM does not allow for external input of other hydrographs and pollutographs to the Storage/Treatment block. This block of the program, in its published form, only accepts input from the Transport block. To model treatment of the modified hydrographs and pollutographs, a special interface routine was developed. The modified hydrographs and pollutographs which account for surface management control techniques were then modelled through treatment.

The SWMM will model various treatment unit operations, including natural sedimentation, but will not model sedimentation assisted by chemical coagulation. The procedure was to model various treatment options including:

1) natural sedimentation,
2) swirl concentrator,
3) swirl concentrator plus natural sedimentation.

In addition, a manual assessment was made of the predicted results to project the increased effectiveness of using chemically-assisted sedimentation.

The hydrographs of flow and the SS and BOD pollutographs predicted by the model are shown on Figures 16, 17 and 18. Hydrographs and pollutographs are shown for the three treatment options described above.

Table 7 summarizes the results of the model simulation, showing the predicted effectiveness of the treatment options modelled, as well as the surface storm water management controls for the one-year synthetic storm following a 30-day dry period.

It is important to note that the storage/sedimentation tank was modelled assuming weir flow and, therefore, the tank remains full after the storm event. The loadings in Table 7 are valid if the contents of the storage tank are not discharged to the river after the storm but, rather, are drained elsewhere, e.g. to the adjacent trunk sanitary sewer. This approach might be satisfactory if this watershed were the last development area within the Rideau River watershed to be considered. The

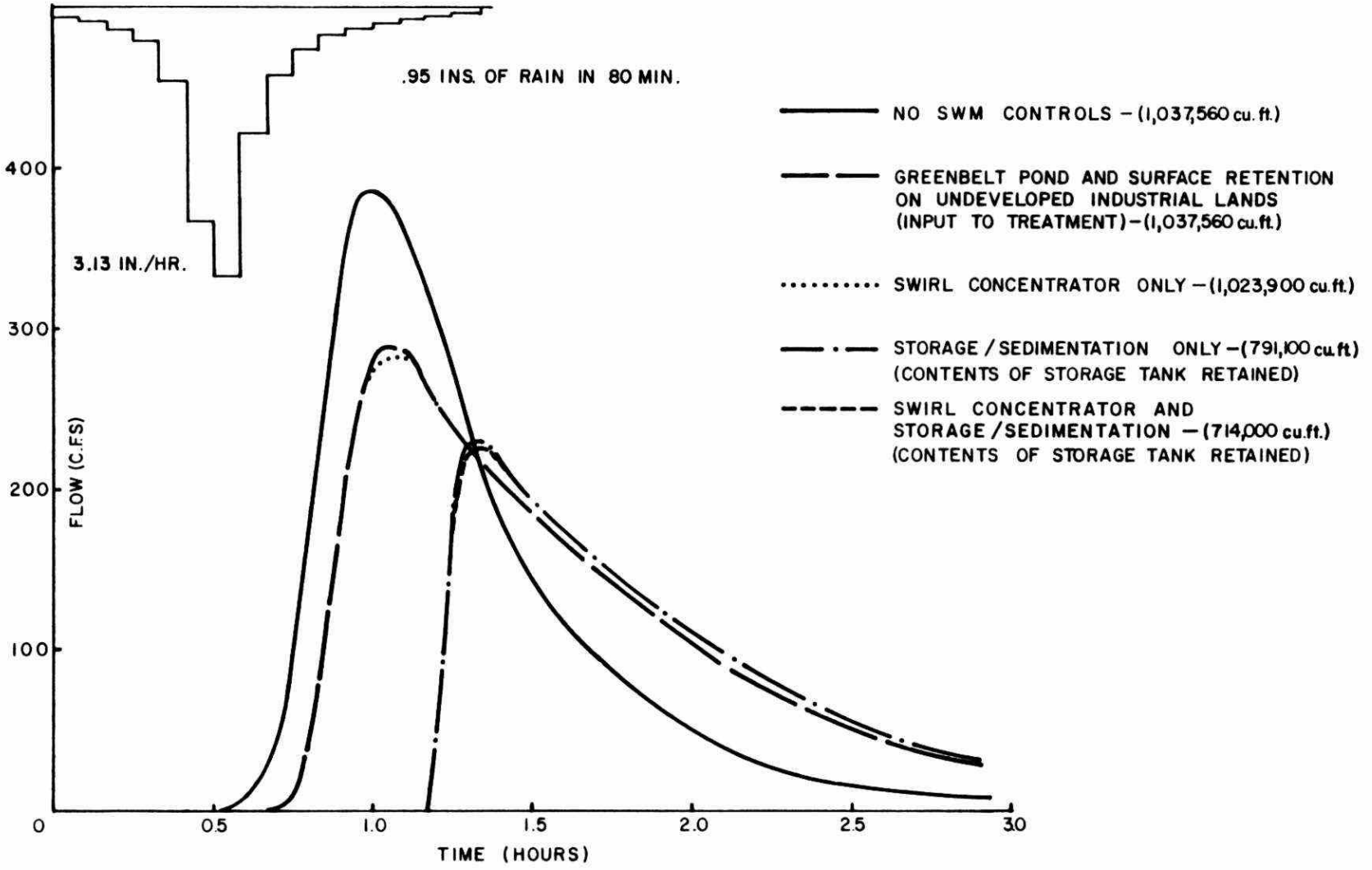


FIGURE 16. HYDROGRAPHS THROUGH TREATMENT

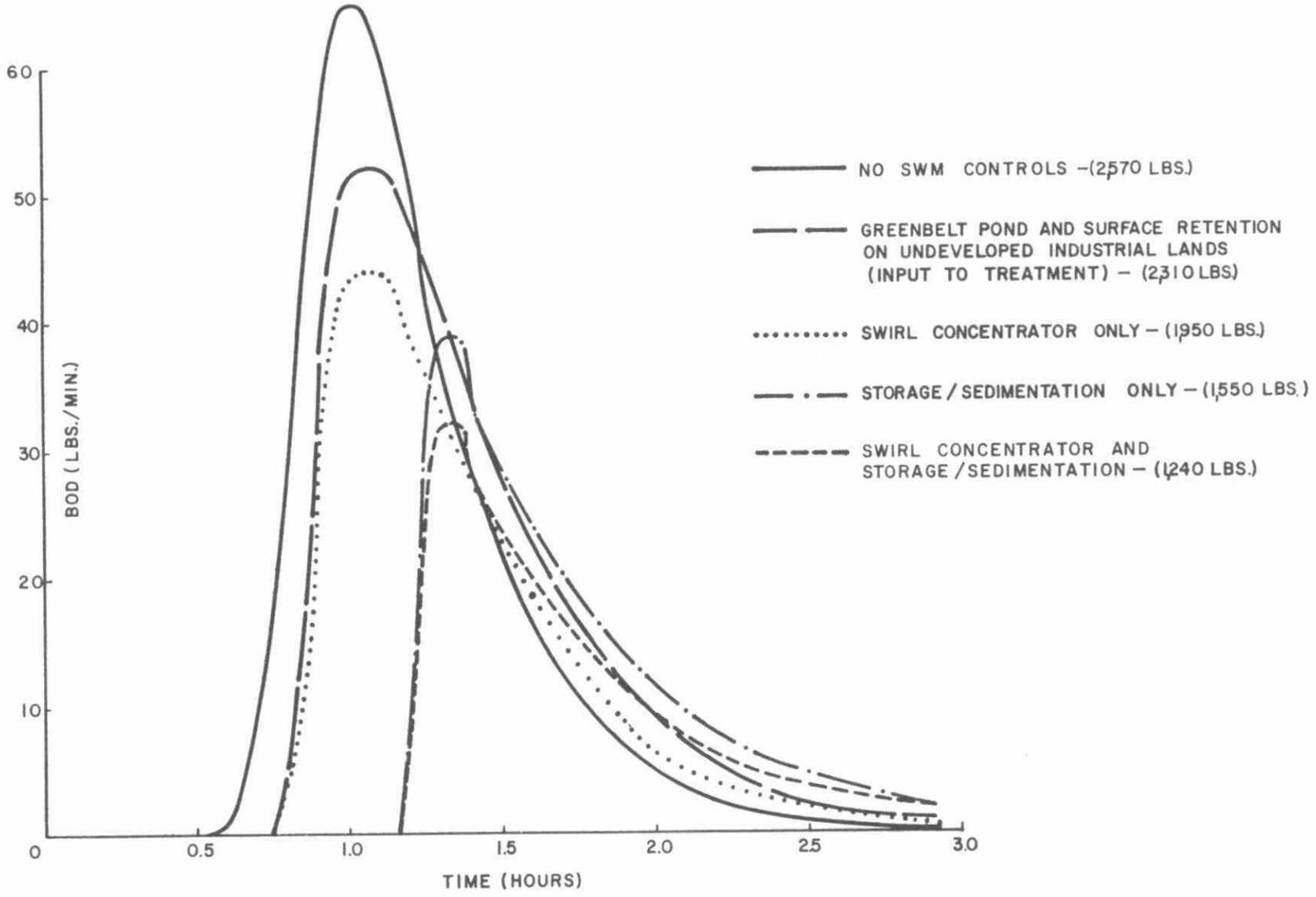


FIGURE 17. BOD POLLUTOGRAPHS THROUGH TREATMENT

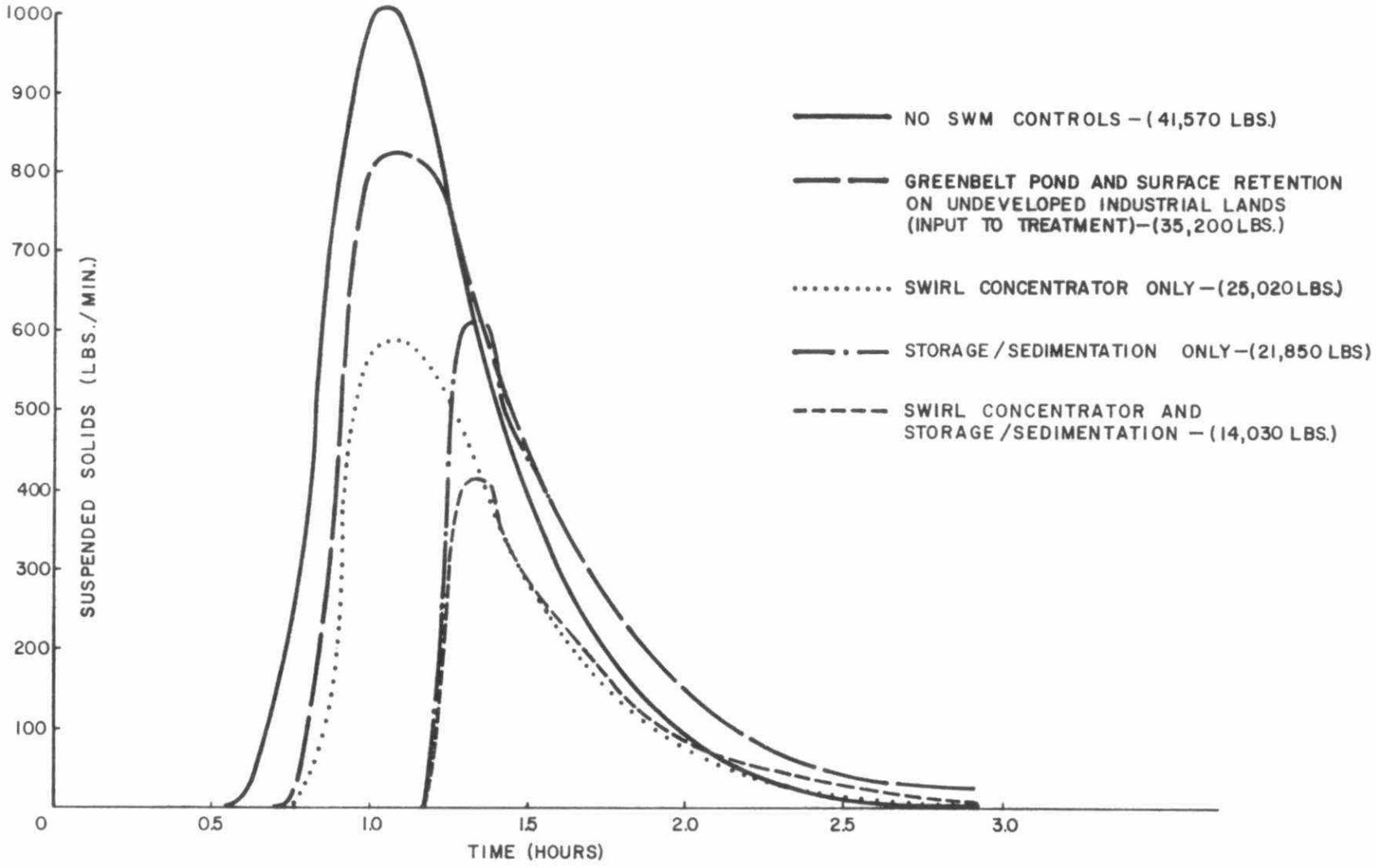


FIGURE 18. SUSPENDED SOLIDS POLLUTOGRAPHS THROUGH TREATMENT

TABLE 7. SUMMARY OF FLOW AND QUALITY RESULTS THROUGH TREATMENT (Contents of Storage Tank Retained)

| Management/Treatment Options | Total Runoff $ft^3$ | Total $BOD_5$ lb | Total SS lb | Avg. $BOD_5$ conc. mg/L | Avg. SS conc. mg/L |
|---|---|---|---|---|---|
| 1) No Controls | 1,037,560 | 2,570 | 41,570 | 40 | 640 |
| 2) SWM Controls | | | | | |
| - surface storage on impervious areas of new development | 1,037,560 | 2,310 (90%) | 35,200 (85%) | 36 (90%) | 544 (85%) |
| - Greenbelt storage pond | | | | | |
| 3) Swirl Concentrator | 1,023,900 | 1,920 (75%) | 24,900 (60%) | 30 (75%) | 390 (60%) |
| 4) Storage/ Sedimentation | 791,100 | 1,550 (60%) | 21,850 (52%) | 31 (78%) | 440 (68%) |
| 5) Swirl Concentrator and Storage Sedimentation | 714,000 | 1,240 (48%) | 14,030 (34%) | 28 (70%) | 310 (48%) |

Merivale watershed, however, is only one of several where development is proposed and, thus, treatment of storm flows through the sanitary system is not considered to be an acceptable precedent.

On the basis of the foregoing, the contents of the storage tank would eventually have to be drained to the Rideau River, increasing the total loadings above those shown in Table 7. Settled solids, of course, would be removed from the tank by mechanical means. Average concentrations for outflow to the Rideau River shown in Table 7 were still considered valid and, on this basis, new total loadings were calculated and are summarized in Table 8.

Comparison of Tables 7 and 8 shows that discharging the supernatant contents of the storage/sedimentation tank after each storm event would add an average of approximately 400 to 500 lb BOD and approximately 5,000 to 7,000 lb SS loading to the Rideau River for such a severe storm.

TABLE 8. SUMMARY OF FLOW AND QUALITY RESULTS THROUGH TREATMENT
(Contents of Storage Tank Discharged)

| Management/Treatment Options | Total Runoff $ft^3$ | Total $BOD_5$ lb | Total SS lb | Avg. $BOD_5$ conc. mg/L | Avg. SS conc. mg/L |
|---|---|---|---|---|---|
| 1) No Controls | 1,037,560 | 2,570 | 41,570 | 40 | 640 |
| 2) SWM Controls | | | | | |
| - surface storage on impervious areas of new development | 1,037,560 | 2,310 (90%) | 35,200 (85%) | 36 (90%) | 544 (85%) |
| - Greenbelt storage pond | | | | | |
| 3) Swirl Concentrator | 1,023,900 | 1,920 (75%) | 24,900 (60%) | 30 (75%) | 390 (60%) |
| 4) Storage/ Sedimentation | 1,037,560 | 2,000 (78%) | 28,500 (68%) | 31 (78%) | 440 (68%) |
| 5) Swirl Concentrator and Storage Sedimentation | 1,023,900 | 1,790 (70%) | 19,800 (48%) | 28 (70%) | 310 (48%) |

It is most important to recognize that both Tables 7 and 8 indicate that $BOD_5$ concentrations predicted in the discharge to the Rideau River would substantially exceed the limit of 5 - 6 mg/L suggested by the Ontario Ministry of the Environment.

To approach the quality criteria suggested by the MOE, a chemical coagulant, such as alum, would be required to assist in sedimentation. As discussed earlier, this would be expected to increase sedimentation efficiency to the extent that SS removal efficiency would increase to approximately 90% and BOD removal efficiency would increase to approximately 70%. Table 9 summarizes the predicted flow and quality characteristics of the discharge to the Rideau River if chemical coagulation were used to assist the sedimentation process.

TABLE 9. SUMMARY OF FLOW AND QUALITY RESULTS THROUGH TREATMENT (Sedimentation with Chemical Coagulation)

| Management/Treatment Options | Total Runoff $ft^3$ | Total $BOD_5$ lb | Total SS lb | Avg. $BOD_5$ conc. mg/L | Avg. SS conc. mg/L |
|---|---|---|---|---|---|
| 1) No Controls | 1,037,560 | 2,570 | 41,510 | 40 | 640 |
| 2) SWM Controls | | | | | |
| - surface storage on impervious areas of new development | 1,037,560 | 2,310 (90%) | 35,200 (85%) | 36 (90%) | 544 (85%) |
| - Greenbelt storage pond | | | | | |
| 3) Storage/Sedimentation with chemical Coagulation | 1,037,560 | 690 (27%) | 3,520 (8%) | 10.7 (27%) | 54 (8%) |

Table 9 shows that chemical coagulation would substantially decrease the predicted pollutant loading to the Rideau River for the synthetic one-year storm to the point where predicted pollutant concentrations in the discharge to the Rideau River approach the limits suggested by the Ministry of the Environment. The addition of a chemical coagulant, however, would be costly and should be considered only as a last resort, if monitoring of effluent from the area ultimately shows that this type of treatment is required.

## 5.5 Quality Parameters Modelled and Accuracy of Predictions

### 5.5.1 General

Assessment of the effects of storm water runoff from urban and rural areas on the receiving waters must involve a number of factors of particular concern, including:

a) public health,

b) stream eutrophication,

c) fish life.

Public health. Public health is of primary concern where lakes and rivers are used for recreation such as swimming. The standards used to measure the safety of a water body were discussed previously in this report and, in general, relate to acceptable levels of total coliform, fecal coliform and fecal streptococcus.

The research data available does not permit accurate predictions of coliform levels in the runoff from either rural or urban lands. Some data is available from studies in a number of cities throughout the United States on the levels of total coliform expected in urban runoff. There is, however, no established relationship between total coliform, fecal coliform and fecal streptococcus, as these vary tremendously and are site specific.

The version of the STORM which was used in this study for long-term continuous simulation does not model coliform bacteria levels. The SWMM, which was used to model the effects of individual events, does make predictions for levels of total coliform but does not attempt to assess fecal coliform or fecal streptococcus.

Since total coliform, and in particular fecal coliform, are the prime indicators of the suitability of water bodies for body contact recreation activities, the usefulness of the model predictions in this regard is somewhat limited. The model predictions do indicate, however, as expected, that coliform levels will, in general, exceed acceptable levels for most conditions of storm water runoff and that adequate facilities will be required to attenuate the high coliform levels. However, a subjective evaluation on the basis of the available literature suggests that there will be a high degree of natural die-off in coliform levels after 24 to 48 hours retention.

Stream eutrophication. Eutrophication as referred to here is the aging of a water body by over-enrichment. This aging is normally accelerated by urbanization because more nutrients are generally discharged to the water courses in the runoff from urban areas than from non-urban areas.

Although both nitrogen and phosphorus are the main nutrients discharged, it is generally conceded that phosphorus is the more important, particularly as related to the presence of algae blooms. The

MOE guidelines discussed earlier in this report only limit phosphorus in the runoff to the receiving waters.

The STORM model simulation undertaken made predictions for phosphorus contribution to the runoff and the phosphorus concentrations in the runoff from the development may be compared with the general guidelines issued by the Ministry of the Environment.

The SWMM makes predictions for phosphorus discharged from each subcatchment in the Runoff block only. Phosphorus is not routed through the system in the Transport block and was, therefore, not included in the description of alternative management and control systems described earlier.

Fish life. The ability of fish to survive in a water body depends to a large degree on the dissolved oxygen levels in the water. One parameter which is used to assess the relative effects of storm water runoff on dissolved oxygen is the biochemical oxygen demand (BOD) created.

Both models used in the simulation predict levels of oxygen demand to be washed from the watershed to the receiving waters. Comparing these predicted values with the limits suggested by the MOE will permit some assessment of the effect of development.

### 5.5.2 Suspended solids

One of the most important parameters in the analysis of urban runoff is the suspended solids associated with the runoff. The characteristics of storm water are such that most pollutant parameters, with the possible exception of coliforms, are associated with the solids content of the runoff. Facilities which will produce an effective removal of solids will usually result in a relatively proportional removal of other associated pollutants.

Much of the research undertaken to date relating to quality of both urban and rural runoff has concentrated on suspended solids and the relationship of suspended solids to other parameters. Thus, the model simulations relating to the quality of runoff are generally based on predictions of suspended solids. Other parameter predictions are then calculated proportionally.

The MOE guidelines do not specifically state the acceptable limits for suspended solids in the runoff from urban areas. They deal rather more specifically with the various parameters affecting public health, stream eutrophication and fish life. The suspended solids content of storm runoff is, however, most important in the overall assessment of a development area. As mentioned above, most of the pollutant parameters of major concern are generally associated with the suspended solids and the removal of the solids from the runoff will have a beneficial effect on the removal of the other pollutants.

## 5.6 Accuracy of Model Predictions

The STORM and SWMM have been extensively tested to determine accuracy of results for quantity and quality of runoff, both in the United States and in Canada. On the basis of the verification undertaken to date on the models, it is expected that quantity predictions, in particular, should be reasonable for average conditions of development under the various land uses.

Runoff calculations are based on sophisticated mathematical procedures considering storage, routing, infiltration and other factors, and are considered to be relatively accurate. Quality predictions are based on average quality parameters measured in a number of municipalities, primarily in the United States, but also in Canada. Actual quality results for the Merivale Area in Nepean could, therefore, vary significantly from those predicted. The model simulation does, however, provide good representation of the relative effects of the various management and treatment options studied.

It is recommended that an extensive monitoring program be established to determine the effects of such controls and to more firmly establish the actual quality of runoff as development and servicing progress. The results can then be compared with the model predictions and assist in the decision-making for possible additional treatment requirements.

## 6. ESTIMATED COSTS

Estimated costs for the storm water management system, as described herein, may be divided into two classifications:

1) surface storage and management,
2) subsurface storage/treatment.

### 6.1 Surface Storage and Management

Surface storage, as described previously, includes both surface storage on impervious areas of new development lands, as well as the storm water retention pond within the NCC Greenbelt lands.

Capital costs for surface controls on impervious areas are considered to be small but, in any event, are the responsibility of the developer. Maintenance costs for such controls are expected to be somewhat more than normal, but again, most of these operating costs are considered to be the responsibility of the individual owners.

The analysis indicates the need for effective street sweeping policies. It is understood that the Township of Nepean is using the more effective vacuum street sweepers. Present practice is to sweep all urban areas approximately four times over the summer months. This results in an average street sweeping interval of about six weeks. To have a significant impact on the pollutant washoff to the Rideau River, a maximum street sweeping interval of approximately two weeks is recommended. It has been estimated that this increase in sweeping frequency would triple present street sweeping costs.

Capital and operating expenditures will be required for pond construction and maintenance. The capital cost of the proposed storm water retention pond in the NCC Greenbelt lands is estimated to be approximately $150,000. In addition, an annual maintenance allowance of approximately $10,000 should be provided for general operation and upkeep of the facility.

### 6.2 Subsurface Storage/Treatment

To utilize the existing trunk sewer pipe storage, it will be necessary to provide a control chamber at the outlet of the pipe system. The estimated cost of this chamber is in the order of $130,000.

A special electrically-operated vehicle with a cleaning bucket will be required to clean the sewer on a continuing basis, particularly after significant storms. The capital cost of such specially-designed equipment is estimated to be approximately $25,000. In addition, an annual operating allowance of approximately $10,000 should be provided for cleaning and maintenance of the trunk sewer.

If additional treatment facilities of the scope discussed earlier become necessary, capital and operating costs could be substantial. Although capital costs may be readily determined, operating costs are difficult to project and will depend on many factors, including:

1) average weather conditions for the particular year,
2) need for chemical coagulant to assist sedimentation,
3) required frequency of cleaning.

There will be operating costs associated with normal clearing of solid deposits from both the trunk sewer facility and the storage treatment tank. If a chemical coagulant such as alum, is found necessary then both the capital and operating costs will increase to include the cost of chemical tank storage and chemicals.

Capital and operating costs for the various treatment facilities described are summarized in Table 10.

TABLE 10. SUMMARY OF CAPITAL AND OPERATING COSTS FOR TREATMENT FACILITIES

| | Treatment Facility | Capital Cost |
|---|---|---|
| 1) | 2 million imperial gallon storage/treatment tank | $3,000,000 |
| 2) | Swirl concentrator | 150,000 |
| 3) | Alum tank storage and feed equipment | 50,000 |

If alum is necessary as a chemical coagulant to assist in sedimentation, it is estimated that the cost of chemicals would be in the order of $2,000 per year. This estimate is based on the present cost of alum applied to the average annual predicted volume of washoff directed into the storage/treatment tank. Additional operating costs would be associated with the cleaning of solids from the tank. This amount will depend on the frequency of cleaning found necessary but will be substantial, particularly if a chemical coagulant is applied.

## 7. CONCLUSIONS AND RECOMMENDATIONS

The study and analysis resulted in a number of conclusions and recommendations, either stated or implied in the report, which are summarized as follows:

1) The application of storm water management controls to provide surface ponding and storage on impervious areas of new development lands would produce beneficial results with respect to reduced loadings to the Rideau River. Although the effect would be most noticeable in the individual areas where the controls are applied, the effect of the reduced loadings from the entire watershed would be significant. The application of these controls to the new development areas (300 acres or approximately 23% of the watershed) would reduce total pollutant washoff from the watershed for a specific storm by approximately 5 to 8%.

It is recommended that these controls be applied and proper management of such controls be enforced for all new developments.

2) The provision of a storm water retention pond to receive the drainage from the Merivale Gardens residential subdivision and the NCC Greenbelt would have a noticeable effect on reduction of pollutant loadings from the watershed. The storm water retention pond would reduce total pollutant washoff from the watershed by approximately 5% to 7%.

It is recommended that the construction of a storm water retention pond within the Greenbelt be considered in developing the sewer system. It is also recommended that the National Capital Commission be approached with respect to property requirements for such a ponding facility.

3) The analysis showed the dramatic effect of pollutant buildup on streets and impervious areas associated with antecedent dry days. Table 6 shows that, for similar storm events, washoff of pollutants is increased approximately three times when antecedent dry days are increased from 17 days to 25 days.

It is recommended that the Township of Nepean review its street sweeping policy with a view to increasing the frequency of sweeping to once every two weeks for this watershed.

4) The pollution washoff loadings predicted for the Merivale watershed were derived from model simulations, using the STORM and SWMM. Calculated loadings, as predicted by the models, are based on average values measured in a number of cities, particularly in the United States. Actual values experienced in the Merivale Area may, therefore, vary significantly from those predicted. The model simulations, however, do provide a good representation of the relative effects and merits of the various storm water management alternatives investigated.

It is recommended that a detailed monitoring system be installed to more accurately assess the magnitude of pollutant washoff from the watershed for the various antecedent conditions and also to assess effects of increased development and extension of servicing. The results of such a monitoring system would provide the necessary data to assist in decision-making for possible additional storage and/or treatment requirements.

5) The analysis showed that a storage/treatment facility with a capacity of two million imperial gallons would contain the runoff from the majority of storm events and provide sedimentation for additional removal of pollutants. It has also been shown that a storage volume of approximately 1.6 million imperial gallons could be provided within the existing trunk sewer for the present scope of development. This available storage will, however, decrease to approximately 0.7 million imperial gallons as development increases and sewer services are extended to provide additional sewer capacity.

It is recommended that the monitoring program include an analysis of the effectiveness of this trunk sewer storage volume. The data from such a monitoring program and analysis would be most useful in determining actual requirements for additional subsurface storage and treatment.

6) The analysis investigated the relative effects of various forms of treatment including:

a) plain sedimentation,
b) sedimentation assisted by chemical coagulation,
c) swirl concentrator,
d) combination of above.

The model simulation showed the relative effectiveness of various treatment options, particularly for SS and BOD removal. The effectiveness of various options on removal of coliforms, however, has not been well documented.

The model simulation provided estimates of total coliform counts predicted in the washoff from the watershed, but the fecal coliform and fecal streptococci associated with such washoff cannot be predicted.

It is recommended that the monitoring program include bacteriological analysis, to more readily assess the magnitude of coliform counts expected and to provide the needed data to assist in decision-making for possible additional treatment requirements.

7) The water quality guidelines set by the Ontario Ministry of the Environment have been formulated in the absence of any regional watershed analysis to assist in establishing total predicted loadings to the Rideau River and the capability of the river to assimilate such loadings. The MOE has suggested that such a study be undertaken to establish a more realistic policy and set of guidelines.

A regional study of the scope as suggested by the MOE would be most desirable to assist in the formulation of a more realistic set of guidelines and criteria for quality of storm water runoff to the Rideau River. It is recommended that funds be established by the municipal and/or provincial governments for such a study.

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